THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

(*FOUNDED* 1908. *INCORPORATED* 1909.)

TRANSACTIONS

AND

NOTES

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OBJECTS OF THE INSTITUTE.

The objects of the Institute are :—

(a) To advance the knowledge of concrete and reinforced concrete, and other materials employed in structural engineering, and to direct attention to the uses to which these materials can be best applied.

(b) To afford the means of communication between persons engaged in the design, supervision and execution of structural engineering works (excluding all questions connected with wages and trade regulation).

(c) To arrange periodical meetings for the purpose of discussing practical and scientific questions bearing upon the application and use of concrete and reinforced concrete and other materials employed in structural engineering.

The Institute is not responsible for the views of individual authors as expressed in Papers, Letters or Notes, but only for such observations as are formally issued on behalf of the Council

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CONTENTS

Discussion at the Meetings : Participation therein by—

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

FIFTH ANNUAL GENERAL MEETING

THURSDAY, MAY 28, 1914

THE FIFTH ANNUAL GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, London, S.W., on Thursday, May 28, 1914, at 4.30 p.m.,

MR. E. P. WELLS, J.P. (the retiring President), in the Chair.

THE SECRETARY (MR. H. KEMPTON DYSON) read the notice convening the meeting, also the report of the auditors, Messrs. Monkhouse, Stoneham & Co., Chartered Accountants, on the balance-sheet, dated December 31, 1913.

The following Report was then submitted :—

REPORT OF COUNCIL FOR 1913-14 SESSION.

The Concrete Institute had on May 14, 1914, 930 Members, 28 Associate-Members, 6 Associates, 54 Students, 5 Special Subscribers, and 16 Honorary Members.

The increase in Membership since the previous figures were given in the Report for the 1912-13 Session is shown in the following table:—

NUMBER OF MEMBERS.

	End of Dec., 1912.	End of June, 1913.	End of Dec., 1913.	May 14, 1914.
Members	877	938	948	930
Associate-Members	—	—	3	28
Associates	—	—	3	6
Students	23	40	47	54
Special Subscribers... ...	8	8	7	5
Hon. Members	—	16	16	16
Total Membership ...	908	1,002	1,024	1,039

Of this total 360 reside in London and its environs, 364 reside in the provinces, and 315 abroad.

An increase in Membership, as was previously reported, occurred upon the extension of the scope of the Institute to deal with structural engineering and the introduction of other improvements. There has been a continued increase during the past Session.

The decision of the Council, that when the total Membership of all classes reached 1,000 an entrance fee of one guinea should be required of Members joining thereafter, has been acted upon. Furthermore, at the beginning of the 1913-14 Session a number of alterations were made in the Articles of Association, with the approval of Extraordinary General Meetings. These alterations, in brief, extended the classification of the Membership by the inclusion of classes of Associate-Members, Associates, and Graduates. No Graduates have yet been admitted pending the establishment of examinations. At the same time the subscription fee for full Members has been raised to

two guineas per annum. Associate-Members and Associates are each required to pay an entrance fee of one guinea and an annual subscription of one guinea. Students will continue to be admitted as at present, without an entrance fee, though they will be required to pay a transfer fee of one guinea upon being transferred from Studentship to Associate-Membership.

The finances of the Institute are shown by the accompanying balance sheet. In the previous Annual Report a surplus was reported and attributed chiefly to the income having been increased abnormally by the collection of a number of subscriptions in arrear. This year the position is reversed, in that a deficit of £161 19s. 10d. has been realized. This has resulted chiefly from the continued growth of the Society. The extra number of meetings held in 1913, the larger Membership, and the changing of the Rules have entailed extra expenditure under the headings of printing, reporting, postage, hire of hall, general expenses, and office salaries, and it was on account of this that the larger rate of subscription and the entrance fees have been imposed. The work of the Concrete Institute compares very favourably with the work of other Societies, considering its very moderate fees.

The number of meetings and educational lectures given in the previous Session was larger than in any earlier Session. In the past Session the increased number has been maintained.

The following is a list of meetings held during the Session :—

SIXTH SESSION.

Educational Lectures.

Wednesdays November 12, 19, 26, December 3, 10, 17, 1913 : A course of Six Lectures on "Reinforced Concrete : Its Commercial Development and Practical Application," by Mr. H. Kempton Dyson, Secretary of the Concrete Institute, Lecturer on Reinforced Concrete and Structural Engineering at L.C.C. School of Building, M.Int.Assn. Testing Materials, etc.

General Meetings.

Thursday, November 13th, Thirty-ninth Ordinary General Meeting: Presidential Address by Mr. E. P. Wells, J.P.

Thursday, November 27th, Fortieth Ordinary General Meeting: Paper by Mr. W. A. Green, M.A., B.Sc.Eng. (St. Andrews), Assoc.M.Inst.C.E. (Member), on "The Differential and Integral Calculi for Structural Engineers."

Thursday, December 11th, Forty-first Ordinary General Meeting: Paper by Mr. W. Laurence Gadd, F.I.C. (Member), on "Some Fallacies in Testing Cement."

Thursday, January 8, 1914, Forty-second Ordinary General Meeting: Paper by Mr. Percival M. Fraser, A.R.I.B.A. (Member), on "Factory Construction."

Thursday, January 29th, Forty-third Ordinary General Meeting: Discussion on Joint Report of the Reinforced Concrete Practice Standing Committee and the Quantity Surveyors' Association on "Standard Methods of Measurement for Reinforced Concrete Work."

Thursday, February 12th, Forty-fourth Ordinary General Meeting: Adjourned Discussion on Mr. Fraser's paper on "Factory Construction."

Thursday, February 26th, Forty-fifth Ordinary General Meeting: Paper by Mr. W. Cyril Cocking (Member), on "Calculations and Details for Steel-frame Buildings from the Draughtsman's Standpoint."

Thursday, March 12th, Forty-sixth Ordinary General Meeting: Paper by Mr. Allan Graham, A.R.I.B.A. (Member), on "Forms for Concrete Work."

Thursday, March 26th, Forty-seventh Ordinary General Meeting: Adjourned Discussion on Mr. W. Cyril Cocking's paper on "Calculations and Details for Steel-frame Buildings from the Draughtsman's Standpoint."

Thursday, April 16th, Forty-eighth Ordinary General Meeting: Paper by Mr. OSCAR FABER, B.Sc., Assoc.M.Inst.C.E., A.M.I.E.E., A.C.G.I. (Member), on "The Design of Steel and Reinforced Concrete Pillars, with special reference to Secondary and Accidental Stresses."

Thursday, April 23rd, Forty-ninth Ordinary General Meeting: Paper by Mr. WILLIAM E. A. BROWN, A.R.I.B.A., M.C.I., on "The Architect and Structural Engineering."

Thursday, May 14th, Fiftieth Ordinary General Meeting: Paper by Mr. JOHN A. DAVENPORT, M.Sc.(Vict.), B.Eng.(Liverpool), Assoc.M.Inst.C.E., A.M.I.Mech.E. (Member), and Professor S. W. PERROTT, M.A.I.(Dublin), M.Inst.C.E., Professor of Engineering at Liverpool University (Member), on "Sand and Coarse Material and Proportioning Concrete."

Thursday, May 28th, Fifth Annual General Meeting and Fourth Annual Dinner.

The thanks of the Institute are due to the authors of papers.

As the result of a ballot among Members of Council the bronze medal for the best paper read in the 1912-13 Session has been awarded to Mr. S. Bylander, for his paper entitled "Steel-frame Buildings in London."

It having been represented that some Members resident in the provinces and abroad, or for other reasons unable to attend the meetings, are desirous of receiving copies of papers read at General Meetings in advance of their publication in the *Transactions*, the Council has decided that in future Members may receive regularly advance copies of papers upon payment of 5s. annually to cover the cost of postage, etc. The privilege of receiving advance copies will be extended without any such payment to all those Members who pay the new rate of subscription in the Membership class—namely, £2 2s. per annum.

The attendance at meetings during the past Session has been higher than in any previous Session, and at

one meeting a record number of Members and Visitors was present. The Educational Lectures were also extremely well attended. The Third Annual Dinner was very successful, the attendance being slightly larger than at the second dinner. Numerous commendations have been received to the effect that the dinner was one of the most enjoyable given by technical societies in the past Session.

In the past Session informal meetings of the Junior Members of the Institute have been instituted. The meetings have so far been held on Friday evenings, the first two taking place on April 3 and May 1, 1914. The attendance at these meetings has been very encouraging, and it is confidently expected that the meetings will be of great educational assistance to Junior Members of the Institute. The meetings will be held once a month during the Session, with a Member of Council in the Chair on each occasion.

In former Annual Reports reference has been made to a "Commtitee appointed to consider the widened scope of the Concrete Institute," the title of this Committee being subsequently changed to the "Improvements Committee." An abstract of the Report of the Committee was appended to the Report of the Council for the 1911-12 Session, in which it was stated that the Committee was appointed to take energetic steps to develop the structural engineering side of the widened scope and that the Committee had defined for the purpose of the Institute that "Structural Engineering" was that branch of engineering which dealt with the scientific design, the construction and the erection of structures of all kinds in any material. The Committee further defined "Structures" as being those constructions which are subject principally to the laws of Statics as opposed to those constructions which are subject to the laws of Dynamics and Kinematics, such as engines and machines. The Committee unanimously recommended—and the Council subsequently adopted their recommendation—that a subtitle should be added to the Institute, so that the full title should be—

THE CONCRETE INSTITUTE

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The Committee also recommended the institution of examinations in structural engineering. Accordingly, the Council during the past Session appointed an Examination Committee, which was subsequently, in December last, merged in a nucleus Examination Board, consisting of Professor Henry Adams, Mr. E. Fiander Etchells, Mr. W. G. Perkins, and Mr. E. P. Wells. The Examination Board has compiled a syllabus and rules for the proposed Examination, which are appended to this Report. It is proposed to hold the first Examination next year. The Improvements Committee collected information in connection with a revision of Rules. The alterations to the Articles of Association previously referred to as having been carried at the Extraordinary General Meetings at the beginning of the Session were made and adopted at the suggestion of the Committee. At the same time proposals were put forward for the amendment of the Memorandum of Association whereby the enlarged scope would be more clearly defined, although the Committee had in their original Report stated their opinion that Clause 3 (1) of the present Memorandum did not limit the scope of the Institute to concrete and reinforced concrete, but that the Clause enabled the Institute to deal with iron (including steel), bricks, gravel, sand, cements, and other structural materials and their application. The amendment proposed to Clause 3 (1) was as follows, the words to be added being shown in black type, and the words to be omitted by italic type within square brackets :—

> 3. (1) To advance the knowledge of concrete and reinforced concrete **and other materials employed in structural engineering**, [*their constituents*], and to direct attention to the uses to which these materials can be best applied.

Consequential alterations were made in other paragraphs defining the objects of the Institute. Also in addition it was proposed to make an alteration, consisting

of adding the words "and Associate-Member" to the word "Member" in Clause 7, which defines the liabilities of the Members, this alteration being suggested in view of the fact that a new class of Associate-Members was created by the alterations to the Articles. When the alterations were duly submitted to the Board of Trade, after their approval by the General Membership, the Institute was required to give, and gave, an undertaking to apply to the Court for allowance of the alterations to the Memorandum. The Board of Trade raised certain objections, however; in particular they objected that the addition of the words "and Associate-Member" was not an alteration of the objects of the Institute, which was the only alteration permissible in the Memorandum of Association, and that only with the approval of the Court. The Institute's counsel has similarly advised the Institute. The Board of Trade raised the further objection that, as the proposed alterations to the objects appeared to have the effect of extending the scope, they were of the opinion that the enlarged scope ought to be shown by an alteration in the title of the Institute. The Institute's counsel has also expressed the opinion that the resolution which was actually passed did not in terms refer to any alteration in the Memorandum of Association, the resolution using the words "New Regulations." Counsel expressed the opinion that the Court might very well take this objection and refuse the order.

Differences have arisen in the Council as to what action the Members should be recommended to take so as to put the matter in order before applying to the Court, and the Council has decided to place the following alternative policies before a General Meeting of the Members before going into Court:—

(*a*) To rescind the alterations to the Memorandum and revert to the original Memorandum of Association with the necessary further alterations to the Articles to provide for Associate-Membership.

(*b*) To rescind the alterations to the Memorandum of Association and to repass the same with such additional alterations as may be required to meet the objections of the Board of Trade.

In connection with the second of these policies, the Council regret to find that a misunderstanding has arisen as to the intention of one paragraph out of twenty-five, namely, Clause 3 (2). If the second policy be agreed to, "concrete and reinforced concrete" will be specially and specifically mentioned in the aforesaid clause. Special meetings for the purpose of deciding between these policies will be convened at the beginning of next Session.

The Council and Committees have been very much occupied in the past Session with technical matters. In previous Reports the action of the Institute has been recorded in respect to the Regulations made under the provision of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete. The Institute made suggestions upon a draft which was submitted by the London County Council, and subsequently made suggestions upon the first set of Regulations when issued by the London County Council. These suggestions for the amendment of the Regulations were sent to the Local Government Board, and in June, 1913, the London County Council rescinded their first set of Regulations and made new Regulations, which were the outcome of prolonged negotiations between technical advisers of the Council and the Local Government Board. The statutory notice of the intention of the London County Council to apply to the Local Government Board for allowance was duly given to the four Societies named in the Act, namely, the Royal Institute of British Architects, the Institution of Civil Engineers, the Surveyors' Institution, and the Concrete Institute. The second (and revised) Regulations thus came before the Societies for consideration with a view to submitting further suggestions for amendment to the Local Government Board. It was found that as a result of the negotiations between the technical advisers of the London County Council and the Local Government Board, many drastic alterations had been made to the first set of Regulations, and, as the Building Acts Committee of the L.C.C. reported, "in some instances they render the Regulations somewhat more onerous than those originally adopted by the Council."

The matter is naturally one of extreme importance to the Members of this Institute, not merely as affecting practice in London, but because the Regulations, when finally approved, will probably be referred to by municipalities in promoting Regulations for their respective localities. The Council and the Standing Committees, *i.e.* the Science Standing Committee, the Reinforced Concrete Practice Standing Committee, the Parliamentary Committee, and the Tests Standing Committee, have therefore given careful detailed consideration to the revised Regulations. That the Concrete Institute has on the whole given prolonged consideration to this matter is shown roughly by the following statistics of the number of hours occupied by meetings of the Council and Committees :—

1910-11 Session: Draft Regulations, 19 hours.
1911-12 Session: First Regulations, 12 hours.
1913-14 Session: Second Regulations, 47 hours.

The Institute's suggestions as to the amendment of the second set of Regulations were sent to the other technical societies, and the Institute has been informed that they have been supported in large part by the Royal Institute of British Architects and by the Surveyors' Institution. The Institution of Civil Engineers informed the Institute that they had not made suggestions for amendment in detail. The Institute's suggestions were finally submitted to the Local Government Board in December last, and are now under consideration by the technical advisers of the London County Council and the Local Government Board.

The Institute's Committees have been so closely occupied with the L.C.C. Regulations that they have not been able to do much other work, though they have a number of subjects in hand for consideration. The details of their work are given below.

In June, 1913, Major H. S. Rogers and Mr. Morgan E. Yeatman were co-opted as Members of Council.

The Committees appointed by the Council for the Session were as follows :—

THE FINANCE AND GENERAL PURPOSES COMMITTEE.

Chairman—Mr. H. D. Searles-Wood.

Ordinary Members—Professor Henry Adams, Sir Henry Tanner, Mr. G. C. Workman.

Also *ex-officio* the President and the Chairman of each Standing Committee.

The Science Standing Committee.

Chairman—Mr. E. Fiander Etchells.

Vice-Chairman—Mr. H. D. Searles-Wood.

Hon. Secretary—Mr. W. G. Perkins.

Ordinary Members—Professor Henry Adams, Mr. H. K. G. Bamber, Mr. William Dunn, Mr. J. E. Franck, Mr. C. F. Marsh, Dr. J. S. Owens, Mr. R. W. Vawdrey, Mr. F. E. Wentworth-Shields, Mr. Morgan E. Yeatman.

Tests Standing Committee.

Chairman—Mr. W. G. Kirkaldy.

Vice-Chairman—Mr. Alexander Drew.

Hon. Secretary—Mr. A. Alban H. Scott.

Ordinary Members—Mr. H. K. G. Bamber, Mr. A. C. Davis, Mr. Percival M. Fraser, Mr. William F. King, Major H. S. Rogers, Mr. R. H. Harry Stanger, Lieut.-Col. J. Winn.

Parliamentary Standing Committee.

Chairman—Mr. H. Percy Boulnois.

Vice-Chairman—Mr. W. H. Johnson.

Hon. Secretary—Mr. J. E. Franck.

Ordinary Members—Mr. H. H. D. Anderson, Mr. T. de Courcy Meade, Mr. J. S. E. de Vesian, Mr. Matt Garbutt, Mr. Osborn C. Hills, Mr. W. G. Perkins, Mr. Edwin O. Sachs, Mr. L. Serraillier, Mr. Henry Tanner.

Reinforced Concrete Practice Standing Committee.

Chairman—Mr. Alexander Drew.

Hon. Secretary—Mr. G. C. Workman.

Ordinary Members—Mr. Ewart S. Andrews, Mr. J. B. Ball, Mr. F. Bradford, Mr. S. Bylander, Mr.

Oscar Faber, Mr. Moritz Kahn, Mr. P. W. Leslie, Mr. J. Petrie, Mr. F. Purton, Mr. Lewis H. Rugg, Mr. A. Alban H. Scott, Mr. H. D. Searles-Wood, Mr. T. B. Shore, Mr. R. T. Surtees, Mr. R. W. Vawdrey, Mr. T. A. Watson.

The President is *ex-officio* Member of all Standing Committees.

In accordance with the Rules of the Institute, one Vice-President has to retire every two years in order of seniority. Accordingly Mr. Edwin O. Sachs retired, and was re-elected a Vice-President in November, 1913.

Mr. E. P. Wells's term of office as President expiring in May, Professor Henry Adams was appointed President for the ensuing two years. The Council is pleased to record that Mr. E. P. Wells will continue as a Member of Council in the capacity of Past-President, and will continue to give the Institute that help and guidance which have been so valuable in the past. The appointment of Professor Adams as President created a vacancy among the Vice-Presidents—who are required to number five—which the Council decided to fill by the appointment of Mr. H. D. Searles-Wood as Vice-President. This will create a vacancy among the ordinary Members of Council.

The following Members of Council have resigned during the past year: Mr. E. J. Lovegrove and Mr. Henry Tanner.

The Council deeply regret to record the decease of Mr. William G. Kirkaldy, an esteemed Member of Council, Chairman of the Tests Standing Committee, and one of the representatives of the Concrete Institute on the Joint Committee on Reinforced Concrete conducted by the Royal Institute of British Architects.

The foregoing vacancies among ordinary members of Council have not yet been filled.

The Council also regret to report the decease of the following Members of the Institute during the past Session :—

Mr. W. John Conn, Executive Engineer, Bilimora Sara Railway, Bombay Presidency, Bilimora, India.

Mr. W. F. Curry, M.R.San.I., District Engineer, P.W.Dept., Middleburg, Transvaal, South Africa.

Mr. J. Wilding Dawson, Assoc.M.Inst.C.E., Assistant Director, P.W. and Survey's Dept., Port Louis, Mauritius.

Mr. E. Vaughan Edmunds, Surveyor to Lansilin District Council, Langollen.

Mr. P. Hanson, Featherstone Road, Southall, Middlesex.

Mr. William H. Johnson, B.Sc., M.Iron and Steel Inst., Vice-Chairman of the Parliamentary Standing Committee, Woodleigh, Altrincham.

Mr. Egbert Rushton, Engineer to U.D.C., Council House, Cleethorpes.

Mr. William Schollar, Half Moon Lane, Herne Hill, S.E.

Mr. James Stewart, M.Inst.C.E., Tua o Rang, Owens Road, New Zealand.

Mr. Paul Wagner, Filzengraben 29-31, Cologne, Germany.

A number of donations to the Library has been received by the Council from individual authors, publishers, and kindred societies, and the Council expresses thanks to the donors. The funds available at present preclude any large purchase of books for the Library. Gifts of books or money for their purchase will be welcomed.

The International Association for Testing Materials having appointed a special Sub-committee to inquire into the reporting of accidents to reinforced concrete buildings, and the collection of statistics thereon, asked the Institute to nominate two representatives for Great Britain. The Council accordingly nominated Mr. H. Kempton Dyson, the Secretary of the Institute, and Mr. S. Bylander to act in that capacity.

The Royal Sanitary Institute having invited the Institute to appoint a delegate at their Sanitary Congress to be held next year at Blackpool, Mr. Searles-Wood was appointed to serve in that capacity.

The Educational Officer of the London County Council having informed the Institute of the facilities offered for instruction at evening classes in London, and asked for advice as to their improvement, was

INCOME AND EXPENDITURE ACCOUNT.

Dr. Year Ending December 31, 1913. *Cr.*

INCOME.	£ s. d.	£ s. d.
To Annual Subscriptions	931 2 10	
„ Special Subscriptions...	31 10 0	
„ Students' Subscriptions	19 8 6	
		982 1 4
„ Sundry Receipts :—		
Various Receipts	9 17 4	
Interest on Deposit Account	11 7 8	
Advertisements	39 4 8	
		60 9 8
„ Balance carried down		161 19 10
		£1,204 10 10
To Balance, being excess of Expenditure over Income 12 months to date, transferred to Surplus Account		161 19 10
		£161 19 10

EXPENDITURE.	£ s. d.	£ s. d.
By Office Salaries	346 10 0	
„ Office Rent, Hire of Hall, Cleaning, Lighting, and Heating	164 10 7	
„ Printing, Reporting, &c.	430 4 7	
„ Postages, Telegrams, &c.	108 12 11	
„ General Expenses	79 0 6	
„ Accountancy	8 8 0	
„ Stationery	46 14 9	
„ Books and Utensils	3 8 6	
		1,187 9 10
„ Amount written off Furniture for Depreciation		17 1 0
		£1,204 10 10
By Balance brought down		161 19 10
		£161 19 10

BALANCE SHEET.

Dr. Year Ending December 31, 1913. *Cr.*

LIABILITIES.	£ s. d.	£ s. d.
To Subscriptions received in advance ...		92 7 6
,, Current Liabilities		259 19 3
,, Surplus :—		
As at December 31, 1912	250 10 3	
Less Excess of Expenditure over Income 12 months to date as per account	161 19 10	
		88 10 5
		£440 17 2

ASSETS.	£ s. d.	£ s. d.
By Furniture :—		
As at December 31, 1912	134 11 6	
Additions—12 months	35 18 4	
	170 9 10	
Less 10% Depreciation written off to Income and Expenditure Account...	17 1 0	
		153 8 10
,, Cash :—		
At Banker's, on Deposit...	250 0 0	
At Banker's Current Account	52 9 6	
	302 9 6	
Less Petty Cash	15 1 2	
		287 8 4
		£440 17 2

We report to the Members that we have obtained all the information and explanations we have required, and that we have examined the above Balance Sheet dated 31st December, 1913, with the Books and Vouchers of the Concrete Institute. We certify that such Balance Sheet is properly drawn up so as to exhibit a true and correct view of the state of the Institute's affairs according to the best of our information and the explanations given us, and as shown by such Books and Accounts.

(*Signed*) MONKHOUSE, STONEHAM & CO.,
Chartered Accountants.

Salisbury House,
London Wall, E.C.
April 7, 1914.

(*Signed*) E. P. WELLS,
President.
H. D. SEARLES WOOD,
Chairman of Finance and General Purposes Committee.
H. KEMPTON DYSON,
Secretary.

informed that the Institute recommended the London County Council to spend more money upon experiments upon structural materials.

Finance and General Purposes Committee.

The Finance and General Purposes Committee has held regular meetings preliminary to each Council Meeting, and the general results of their deliberations are contained in the foregoing particulars of the Council's work for the year.

Science Standing Committee.

In addition to considering the L.C.C. Regulations for Reinforced Concrete the Science Standing Committee has been concerned with the revision of the Standard Notation for Structural Engineering Calculations in view of the criticisms made at the General Meeting when the draft report on the matter was submitted. The finally revised notation will be issued shortly. In conjunction with the Reinforced Concrete Practice Standing Committee a Standard Specification for Reinforced Concrete Work has been prepared in draft and will be submitted for discussion at a General Meeting next Session.

The Science Standing Committee has the following further matters under consideration :—

1. Standardization of joints and connections in reinforced concrete.
2. Advice to superintendents of reinforced concrete work.
3. Amendment of the standard specification for cement.
4. Co-ordination of the standard specification for structural steel of all kinds.
5. The adhesion of and friction between concrete and steel.
6. Reinforced concrete piles.
7. The effect of sewage upon concrete.
8. The effect of oils and fats on concrete.

Reinforced Concrete Practice Standing Committee.

During the past Session the Reinforced Concrete Practice Standing Committee has met, in conjunction with the other Standing Committees and the Council, to consider the L.C.C. Regulations for Reinforced Concrete Work. The Committee has held joint meetings also with delegates of the Quantity Surveyors' Association and with Members of the Concrete Institute who are Quantity Surveyors by profession. Several meetings were held, as a result of which a draft report on a Standard Method of Measurement for Reinforced Concrete was submitted for discussion at a General Meeting. Since this meeting the report has been considered by the National Federation of Building Trades Employers of Great Britain and Ireland and by the Institute of Builders, who have made suggestions for its amendment in certain particulars. Meetings will be subsequently convened to consider the various criticisms and steps taken to issue a final report in due course. The Committee has prepared in conjunction with the Science Committee a draft Standard Specification for Reinforced Concrete Work as recorded above.

The Reinforced Concrete Practice Standing Committee has the following further matters under consideration :—

1. Advice to clerks of works, inspectors, and foremen as to methods of properly executing concrete and reinforced concrete work and of preventing defects and failures.
2. Regulations, recommendations of joint committees, and various methods of calculation in respect to the design of reinforced concrete and the like.
3. Forms and centering for reinforced concrete work.
4. Standard concrete mixtures for general purposes.
5. The use of cinder, ash, clinker, and breeze in concrete.
6. Means of keeping reinforcements in place when concreting.

7. Methods of making concrete watertight and of waterproofing concrete.

TESTS STANDING COMMITTEE AND PARLIAMENTARY STANDING COMMITTEE.

The Tests Standing Committee and the Parliamentary Standing Committee have held joint meetings with the Council and the other Standing Committees, as previously mentioned.

The Tests Standing Committee has the following matters under consideration :—

1. The effect upon steel of the presence of sulphur in aggregates.
2. The grading of aggregates.
3. The expansion and deterioration of concrete due to changes of atmospheric temperature.
4. The effect of the use of sodium silicate on the surface of concrete as affecting reinforcing metal.
5. The erratic results obtained by the Vicat needle in ascertaining the initial setting time of cement.
6. The collection of data regarding the elastic moduli of concrete for stresses within working limits.

INVESTIGATION COMMITTEE.

The Investigation Committee has held meetings at which have been considered (*a*) the action of the Local Government Board in respect to the periods allowed for the repayment of loans sanctioned by them to Local Authorities for the construction of works of reinforced concrete, and (*b*) reports of failures of reinforced concrete structures. In connection with the latter, a Standard Report Form for the use of observers of defects and failures has been drawn up.

JOINT COMMITTEE ON LOADS ON HIGHWAY BRIDGES.

The Joint Committee on Loads on Highway Bridges, conducted by the Concrete Institute, has been engaged in the drafting of their report, but it is not yet completed. It is expected that it will be ready for presentation for discussion at a General Meeting next Session.

APPENDIX

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

RULES AND SYLLABUS OF THE EXAMINATION

BOARD OF EXAMINERS

Superintending Examiner :

*Henry Adams, M.Inst.C.E., M.I.Mech.E., F.S.I., F.R.San.I., President Concrete Institute, etc., late Professor of Engineering at the City of London College.

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*S. Bylander, Past-Chairman J.I.E., M.C.I.

J. D. Cormack, D.Sc., M.Inst.C.E., M.I.Mech.E., M.C.I., Professor of Mechanical Engineering, University College, London.

*E. Fiander Etchells, F.Phys.Soc., M.Math.A., A.M.I.Mech E., M.C.I.

Oscar Faber, B.Sc., Assoc.M.Inst.C.E., A.M.I.E.E., A.C.G.I., M.C.I.

S. M. Dixon, M.Sc., M.A., M.Inst.C.E., M.C.I., Professor of Civil Engineering, Imperial College of Science and Technology.

B. L. Hurst, Assoc.M.Inst.C.E., M.C.I.

H. Lapworth, D.Sc., F.G.S., M.Inst.C.E.

Alan E. Munby, M.A., F.R.I.B.A.

J. S. Owens, B.A., M.D., Assoc.M.Inst.C.E., M.R.San.I., F.R.G.S., F.G.S., M.C.I.

*W. G. Perkins, M.C.I., District Surveyor for Holborn.

*H. D. Searles-Wood, F.R.I.B.A., M.C.I.

*E. P. Wells, J.P., Past-President Concrete Institute.

*M. E. Yeatman, M.A., M.Inst.C.E., M.Am.Soc.C.E., M.C.I.

A. R. Sage, M.C.I., Assistant Principal of London County Council School of Building.

* Executive Committee.

RULES OF THE EXAMINATION

The Examination is divided into two parts :—

Part I. Scientific (for Graduateship) consists of written papers dealing with the scientific basis of the subject.
Part II. Technical (for Associate-Membership) consists of written papers and *vivâ-voce* examinations, dealing with the technology of the subject.

The Examination is held half-yearly in May and October.

1. The age of the candidate at the date of examination is not restricted.

2. A candidate may enter for Part I alone, and if successful he may take Part II at a subsequent Examination. A candidate is not permitted to present himself for Part II unless he has passed Part I, or one of the Examinations (see below) which are accepted by the Council in lieu of Part I ; but a candidate may enter for Parts I and II together.

In Part II a candidate must enter for two at least of the subjects, one of which must be Structural Engineering. The subject or subjects in which he is successful will be named on the Certificate.

3. The Examination is confined to Students and Graduates of the Institute.

4. Candidates exempted from Part I may be registered as Graduates without passing that part of the Examination, but no Certificate will be issued.

5. Applications to be admitted to either or both parts of the Examination must be made on the prescribed form, must be received by the Secretary not later than one month before the date of the Examination, and must be accompanied by the necessary fee.

6. The fee for either part of the Examination taken separately is £1 1s. (One Guinea). If the two parts are taken at one time the fee for the whole Examination is £1 11s. 6d. (One and a Half Guineas).

7. Each applicant will be informed when he has been accepted as a candidate, after which the fee will be forfeited if he does not present himself at the examination for which he has entered.

8. Every candidate who qualifies in, or is exempted from Part I of the Examination, will, on passing Part II, be granted a Certificate of having passed the Associate-Membership

Examination. On passing Part I a Certificate of having passed the Graduateship Examination will be granted.

9. The fact of passing the Examination does not exempt a candidate from the other requirements for election in accordance with the Articles and By-laws of the Institute.

10. Candidates are required to attend at the Examination Room fifteen minutes before the hour fixed for the first paper to be taken by them.

11. Drawing and mathematical instruments, including slide-rules, may be used. The use of books will not be allowed in Part I, but in Part II candidates may bring and use text-books and note-books.

The following will be exempt from the requirement to sit for Part I of the Examination.

- (*a*) Bachelor of Science.
- (*b*) Bachelor of Engineering.
- (*c*) Associate Member of the Institution of Civil Engineers (by examination).
- (*d*) Associate Member of the Institution of Mechanical Engineers (by examination).
- (*e*) Associate of the Royal Institute of British Architects (by examination).
- (*f*) The holder of a commission in the Royal Engineers.
- (*g*) The holder of such other degree or qualification as the Council may determine in specific cases.

SYLLABUS OF THE EXAMINATION.

PART I. (*A*) Compulsory Subjects.

1. PRINCIPLES OF STATICS AND THEORY OF STRUCTURES.

Forces acting on a rigid body; composition and resolution of forces; couples; moments of forces; conditions of equilibrium, with application to loaded structures. Graphical and analytical treatment of the foregoing. Centre of gravity; specific gravity.

Graphic and analytic methods for the calculation of bending moments, shearing forces, and the stresses in individual members of framed structures loaded at the joints; reciprocal diagrams; incomplete frames and redundant members; buckling of struts; effect of different end fastenings on their resistance; combined stresses; section modulus; methods of dealing with statically indeterminate problems, as beams supported at three points, etc.; travelling loads; rigid and

hinged arches ; stresses due to weight of structures ; theory of earth pressure and of foundations ; stability of masonry and brickwork structures.

2. Strength and Elasticity of Materials.

Physical properties and elastic constants of cast iron, wrought iron, steel, timber, stone, concrete, cement, and other materials ; relation of stress and strain ; limit of elasticity ; yield-point ; Young's modulus ; coefficient of rigidity ; extension and lateral contraction ; resistance within the elastic limit in tension, compression, shear, and torsion ; strength and deflection in simple cases of bending ; beams of uniform resistance ; reinforced concrete beams.

Ultimate strength with different modes of loading ; plasticity and permanent set ; working stress ; phenomena in an ordinary tensile test ; stress-strain diagrams ; suddenly applied and impulsive loads ; resilience ; fatigue of metals ; effects of hardening, tempering, and annealing.

Forms and arrangements of testing machines for tension, compression, torsion, and bending tests ; instruments for measuring extension, compression, and twist ; forms of test-pieces and arrangements for holding them ; methods of ordinary commercial testing ; percentage of elongation and contraction of area ; test conditions in specifications for the principal materials of construction.

Part I. (*B*) Selective Subjects.

Two of the following subjects must be taken in addition to the compulsory subjects :—

3. Chemistry.

Constitution of matter ; chemical elements ; Dalton's atomic theory ; Newland's law of octaves ; Mendeleeff's law of periodicity ; modes of chemical action ; atomicity ; analysis and synthesis ; composition of materials employed in structural engineering.

4. Physics. (*Note.*—A candidate taking this subject must be prepared to answer questions in three of the five sections.)

Sound.—Nature of sound ; pitch, intensity, and timbre ; transmitting media ; velocity of sound ; sound waves ; vibrating strings, plates, and membranes ; resonance ; interference ; reflection and absorption of sound.

Light.—Theories of light; transmitting medium; velocity of light; solar spectrum; laws of reflection; photometry; candle-power; candle-feet; absorption of light; colour; polarized light; action of lenses; telescope and microscope.

Heat.—Sensible and latent heat; thermometers; pyrometers; effect of change of temperature in solids, liquids, and gases; transfer of heat; radiation; conduction and convection; relative conductivity; thermal units; Joule's equivalent; thermal capacity; specific heat; combustion.

Magnetism.—Magnets; magnetic phenomena; magnetic field; polarity; the mariner's compass; magnetic meridian; deviation and declination of the compass; inclination or dip; induction; galvanometers.

Electricity.—Static and voltaic electricity; induction; conductors; electro-negative and electro-positive elements; electrolysis; lightning; system of electrical transmission; electrical units; measurement of electrical work; Ohm's law; principles of arc and incandescent lighting.

5. Hydraulics.

Pressure on surfaces; centre of pressure; strength and stability of structures supporting water pressure; laws of fluid friction; impact of water on surfaces; storage of water and construction of reservoirs.

6. Geology.

Classification of rocks; succession of strata in aqueous formations; explanation of geological terms; glacial drift; conditions of deposition in fresh and sea water; denudation; disintegration and chemical decomposition of rocks; method of dealing with bad ground for engineering works.

7. Geodesy.

The theory, structure, and adjustment of the principal surveying and levelling instruments, and the principles of their employment under various conditions; land surveying; contouring; levelling and use of theodolite.

Part II. Technical.

Subject 8 must be taken by all candidates, and at least one other subject.

8. Structural Engineering (Generally).

Materials of construction; loads—dead loads (distributed and concentrated), live loads (rolling and suddenly applied);

bending moments ; resistance moments ; stresses and strains ; shear stresses ; deflection ; secondary stresses ; fatigue of metals ; safety factors ; wind pressures ; standard sections of rolled steel ; properties of sections ; girders—rolled sections (simple and compound), plate web and lattice web, trussed frame ; pillars, columns, stanchions, piers and struts generally ; eccentric loading ; fixity of ends ; roofs—symmetrical and unsymmetrical trusses ; connection of parts ; bridges—girder, suspension, cantilever ; arches—elastic rib, rigid and braced, two and three pivoted ; methods of erection ; testing and inspecting materials of construction ; effect of workshop processes on steel ; mass retaining walls and their stability against water pressure and earth pressure ; pressures in silos, bins, and hoppers.

9. Reinforced Concrete Construction.

General principles ; advantages and disadvantages of reinforced concrete ; materials of construction and their testing ; nature and properties of materials for concrete ; mixing concrete by hand and machine ; effect of frost and precautions to obviate damage ; laying concrete ; testing actual concrete used ; testing completed structures ; failures and causes ; comparison of cost with other methods of construction ; fire-resisting properties ; causes affecting expansion and contraction ; surface finish ; durability and maintenance ; form work (centering, shuttering, strutting, moulds, etc.), precautions in fixing, order and periods of removal.

Routine of designing ; arrangement of roof and floor slabs, cross beams, main beams, and pillars ; loads on floors ; calculation of reinforcement for various parts ; loads on foundations ; rough estimates of cost ; rules and regulations.

10. Steel Frame Construction.

Order of procedure in designing ; external forces, wind, snow, etc. ; arrangement of roofs ; loads on floors ; arrangements of main and cross girders and stanchions ; caps and base plates ; grillages ; junctions ; erection ; brick and stone panels and casings ; protection against fire ; painting ; rough estimates of cost ; rules and regulations.

Candidates desirous of being examined in Masonry, Bridgework, or any other branch of Structural Engineering not specified above may be so examined subject to the approval of the Examination Board and upon notifying the Secretary at least six weeks beforehand.

THE CONCRETE INSTITUTE.

Recd........................

APPLICATION TO ATTEND

PART......OF THE EXAMINATION.

To the COUNCIL OF THE CONCRETE INSTITUTE.

Name in full. I.. ....

Address. of..

...............aged......born the......day of......................1......

at present occupied as ....

hereby apply for the permission of the Council to present myself for Part.........of the Examination, and herewith tender the fee of.........guineas in accordance with Rule 6 of the Examination.

The optional subjects I select to be examined in are—

In Part I* (1)..

(2)..

In Part II. ....

Signature of Candidate..

Address ....

....

Date......................19......

* If "Physics" be selected the three sub-sections chosen should be named.

INSTRUCTIONS TO CANDIDATES AT EXAMINATIONS.

A Candidate may only refer during Part II of the Examination to those books he has personally brought. No borrowing or lending will be permitted.

Answers are to be given on one side only of the paper provided for that purpose, and no notes are to be made in the margins. Rough calculations may be made on the back of the sheets ; no scrap paper will be allowed.

No explanation is allowed to be asked for or given with regard to any question in the Examination Paper, and Candidates are not allowed to speak to each other.

Each answer, preceded by the number of the question, must commence on a separate sheet of paper, and each sheet must have the Candidate's distinguishing number on the top right-hand corner, but his name must not appear anywhere on the paper.

The sheets are to be fastened together at the top left-hand corner in their proper order with the name of the subject written at the head of the first sheet.

A Candidate leaving the Examination Room for any purpose whatever during the Examination may be disqualified.

Each Candidate will be informed by post whether he has passed or failed at the Examination. No other information will be given.

DISCUSSION.

THE PRESIDENT :—In submitting the annual report and balance-sheet I suppose we may take them as read, as I do not want to inflict myself on you at this late hour. The report is an extremely full one, and it deals with the general work that has been carried on by the Concrete Institute during the last twelve months ; and I think every one must admit that we have had an extremely busy time, not only with the ordinary work of the Institute, but also with reference to the new regulations upon reinforced concrete, which you will see by the report have occupied us for many hours. (We have as a matter of fact sat a great many times until half-past ten at night.) I believe they will become law in a very short time, and I am afraid that for most of us who are interested in reinforced concrete we shall find that they are

very stringent indeed. There is a lot of other work to be carried on in the next year that we have had to defer entirely owing to the amount of time that has been spent on these regulations, so that now they are all completed, so far as the Institute is concerned, I think you will find in the next year that we shall be able to issue a lot of useful information to the members and to the world at large; this you will find to be especially the case in reference to the matters mentioned on page 13 of the report. With reference to the balance-sheet, I do not think there is much to say. You will see that this year there is a balance of expenditure over income amounting to £161 19s. 10d., but at the same time we have a fairly large balance in hand at the end of the current year, amounting to £287 8s. 4d. The reason for this is that a large amount of back subscriptions and other things have been included in the accounts; and I trust that this year, with the increased membership, and owing to the new members having to pay an entrance fee and also two guineas per annum, we shall have a credit on the right side by the time the year is closed, without in any way entrenching upon our deposit account, which you will notice is £250. I move that the report and balance-sheet be and are hereby adopted.

SIR HENRY TANNER, C.B., I.S.O., F.R.I.B.A., F.S.I. (Past President C.I.):—I have great pleasure in seconding your motion that this report and balance-sheet should be adopted. It appears to me that the Council should be congratulated on the number of papers which have been read, which must add much to the usefulness of the Institute, and also upon the increase of work which they have done in the past year. I do not think this could have been the case unless we had added some other form of construction besides reinforced concrete. There is one other point I should like to refer to, and that is the financial situation. Of course, I suppose we have had some undue expenditure during the past year, otherwise we should have been in a better condition than we are in now, with £161 to the bad on the year's working. We should not have entrenched upon the previous

year, but it cannot be helped under the circumstances. I am, however, glad to hear, Mr. President, that you think that, proceeding as we are, we shall be in a better condition at the end of the next financial year. I have great pleasure in seconding your motion.

Mr. E. O. SACHS, F.R.S.Ed. (Vice-President C.I.) :—At a short and formal meeting of this description, with merely a quorum present, I do not wish to say much regarding the annual report that will occupy your time to any great extent, but I would in the first place like to congratulate the Institute on the excellent work which it has done in connection with the London County Council Reinforced Concrete Regulations. The work has been most studiously, carefully, and painstakingly undertaken, and we all hope that there will be a successful issue to what has been done in that direction. I also think it is a matter of congratulation that the papers have been more numerous and that some of them have been so very interesting.

The only point that I would like to raise by way of criticism is the regrettable part of the report that devotes several pages to what I will term the re-organization scheme. It strikes me—and I speak on behalf of quite a large number of members—that we have been perhaps trying to go a little too fast and that much could have been avoided by going slow—that is to say, much that appears to be unfortunate could have been avoided, for there has been not only a materially attempted change of policy, but that change has also, unfortunately, been accompanied by what I will term some errors, or misunderstandings, or even a little muddling, which has not conduced to the prestige of the Concrete Institute.

We all hope—and I would here again emphasize the fact that I speak on behalf of a large number of members—that the coming year will find some way out of these little troubles, that compromise will be found to be the suitable way out, and that Mr. Henry Adams in his new year of office may not have a troublous but a pleasant time to look back upon when he ends the very difficult work of presiding over the next year's meetings of the Concrete Institute.

MR. H. D. SEARLES-WOOD, F.R.I.B.A. (Vice-President C.I.) :—I should like to echo Mr. Sachs's sentiment and hope that with a little consideration we may resume our harmonious meetings. When we were learning our Latin grammar we used to repeat—

Amantium iræ amoris integratio est
(The falling out of faithful friends renewing is of love)

and this little bickering between us may only strengthen our admiration for each other, and if we do work together with a will I am sure we shall pull the whole thing through.

The resolution was carried unanimously.

THE PRESIDENT :—The next business is to present the report of the scrutineers on the annual election of members of the Council, as follows :—

We, the scrutineers to the Council Election of the Concrete Institute, have found as follows :—

Yeatman, M. E.	185
Etchells, E. F.	182
Petre, J.	179
Kirkaldy, W. G.... ...	178
Faber, O.	172
Watson, T. A.	156
Rogers, H. S.	125
De Vesian, J. S. E. ...	86

Two hundred and sixteen voting papers were received.

Eighteen lists of attendances were returned instead of voting papers.

There was one spoiled paper.

The votes of two persons had to be ignored owing to their subscriptions not having been paid.

Four papers were rejected owing to the envelopes not being signed.

Yours truly,
(Signed) R. H. LACY.
M. BÉHAR.

THE PRESIDENT :—The next business is the election of auditors, and as this is a matter which must

be proposed by some ordinary member I will ask Mr. Allan Graham if he will kindly do so.

MR. ALLAN GRAHAM, A.R.I.B.A. :—I beg to propose that Messrs. Monkhouse, Stoneham & Co., Chartered Accountants, continue as auditors for this year at a fee of five guineas.

MR. S. BYLANDER :—I will be pleased to second that.

The motion was agreed to.

THE PRESIDENT :—The next business, which is of a very pleasing nature, that I have before me is the presentation of a bronze medal to Mr. S. Bylander for the best paper that was read in the session 1912-13. I think it is hardly necessary for me to make many remarks about Mr. Bylander's qualifications. I think you all know him very well indeed. The paper that he read before us was a most excellent one, and especially as it was the commencement really of our departure in structural engineering. With these few remarks, I now ask Mr. Bylander to kindly step up and receive the medal, and I trust that whenever you look upon the same it will bring back to you pleasant recollections of your time at the Concrete Institute, and especially the time when you read the paper before its members.

MR. S. BYLANDER, having received the medal, said :—Mr. President and gentlemen, it is a surprise to me that you have honoured me with a medal for the paper I have read. I can only assure you, and repeat what the President has said, that it is a pleasure to read a paper before this Institute. I am very proud indeed that you should allow me to read a paper, and I am more proud still that you should honour me with this medal. I have at all times considered that the Concrete Institute is doing a real good thing for the engineering profession at large, and for reinforced concrete and steelwork in particular, and I think that it has a great future before it. I was particularly pleased to lay before you my humble ideas, and I thank you very heartily for the great honour you have done me.

THE PRESIDENT :—The last important business on the paper before us to-night is for me to vacate this Chair after being in it for two years, and to instal Professor Henry Adams as the President of the Concrete Institute for the next two years ; but before doing so I wish to thank you all for the extreme kindness you have shown to me during my two years of office, and I have to thank the Secretary also for the able manner in which he has carried out all his work, and also the staff generally. You know perfectly well what the Secretary has had to do during the last two years, especially in regard to all the various regulations, specifications, etc., that we have had before us. I will now, after these few remarks, ask Professor Adams to take the Chair, and I am afraid in doing so I am sentencing him to two years' hard labour. I see he smiles, so there is no doubt about it that he appreciates it. I trust sincerely that he will have good health to carry out his duties, and that during his term of office the Institute will progress rapidly.

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I. Mech.E., F.S.I., M.S.A., etc. (Vice-President C.I.), then took the Chair. He said :—Mr. Wells and gentlemen, I need hardly assure you that I fully appreciate the great honour that has been conferred upon me, and that I shall use my best endeavours to serve you faithfully and impartially so long as I occupy this Chair. Whatever my personal opinions may be upon any matter that comes before the Institute, I shall feel in duty bound not only to loyally support but to give effect to the best of my ability to the wishes of the majority, and I hope that in a very short time we shall be able to find some method by which the members all over the world will be able to have a voice in the management of the Institute.

MR. H. D. SEARLES-WOOD :—I wish to propose a very hearty vote of thanks to our outgoing President for his able conduct in the Chair during the past two years. I think we have been most lucky in the Presidents we have had up to the present time, and that Mr. Wells has carried on the good tradition which was started by our first Presidents. If I had

to choose a motto for Mr. Wells, I think I should recommend " Experientia Docet," adopting the schoolboy translation : " Docet," he doses ; " experientia," with experience. I am quite sure that by that treatment we have all been very much benefited.

Mr. M. E. YEATMAN, M.A., M.Inst.C.E., M.Am.Soc.C.E., M.C.I. :—I have much pleasure in seconding the vote. Mr. Wells has had stormy times to go through in a few recent meetings, but I hope that he is none the worse for that and that he will survive to enjoy his well-earned retirement, and that in the position of Past-President he will continue to give us his able counsels.

The motion was carried by acclamation.

Mr. E. P. WELLS :—I thank you sincerely for the kind words you have said and for the way the motion has been received, and I can assure you that in the future, as in the past, I will use my very best endeavours on behalf of the Institute. If I can be of any use, you will always be able to command me.

FOURTH ANNUAL DINNER

Thursday, May 28, 1914

THE FOURTH ANNUAL DINNER of the CONCRETE INSTITUTE was held at the Connaught Rooms, London, on the evening of Thursday, May 28, 1914, Professor Henry Adams (President) in the Chair. There were 102 present, and a musical entertainment was given.

List of those present :—

In the Chair.

THE PRESIDENT (Professor Henry Adams, M.Inst.C.E., etc.).

Guests of the Institute.

Mr. Horace L. Boot, M.I.Mech.E., etc., President, Institution of Municipal Engineers.

Mr. C. McArthur Butler, F.C.I.S., Secretary, Society of Architects.

Mr. Thomas J. Carless, M.C.I., President, Quantity Surveyors' Association.

The Hon. Sir John A. Cockburn, K.C.M.G., M.D., late Premier of South Australia.

Mr. W. Curtis Green, F.R.I.B.A., President, Architectural Association.

Mr. George Hubbard, F.S.A., Vice-President, Royal Institute of British Architects.

Mr. Ellis Marsland, M.S.A., District Surveyor for Camberwell, President, District Surveyors' Association.

Mr. F. W. Rice, President, Institute of Builders.

Mr. Leslie S. Robertson, M.Inst.C.E., Secretary, Engineering Standards Committee.

Representatives of *The Builder*, *Concrete and Constructional Engineering*, *The Concrete Institute Transactions*, and the Central News.

MEMBERS.

Mr. John C. Ambrose, Assoc.M.Inst.C.E. ; Mr. H. Woodward Aston ; Mr. H. K. G. Bamber, F.C.S. ; Mr. Walter J. R. Barker, Licentiate R.I.B.A. ; Mr. Habib Basta, Assoc.M.Inst.C.E. ; Mr. Maurice Béhar ; Mr. Percy J. Black, District Surveyor for Wandsworth East ; Mr. George B. Blyth ; Mr. F. Bradford ; Mr. Alfred Brooks ; Mr. A. Stewart Buckle, M.S.E. ; Mr. D. B. Butler, Assoc.M.Inst.C.E., F.C.S. ; Mr. J. F. Butler, Assoc.M.Inst.C.E. ; Mr. S. Bylander ; Mr. F. Dare Clapham, F.R.I.B.A. ; Mr. W. Cyril Cocking ; Mr. R. E. Eiloart ; Mr. C. Fiander Etchells, F.Phys.Soc., M.Math.Assoc., A.M.I.Mech.E., Hon. A.R.I.B.A. ; Mr. J. Ernest Franck, A.R.I.B.A. ; Mr. S. Alfred Frech, Assoc.M.Inst.C.E. ; Mr. George H. Gascoigne, Mr. Allan Graham, A.R.I.B.A. ; Mr. Osborn C. Hills, F.R.I.B.A., District Surveyor for Strand ; Mr. Ernest Hirsch ; Mr. L. Lister Kaye ; Mr. H. M. Kelk ; Mr. W. H. Lascelles ; Mr. W. J. Leahy, Licentiate R.I.B.A. ; Mr. P. W. Leslie ; Mr. W. S. Napier ; Mr. Stanley V. Nicholson ; Mr. W. G. Perkins, District Surveyor for Holborn ; Mr. J. F. Plaister ; Mr. Frederick Purton ; Mr. S. G. Robinson, M.I.Mech.E. ; Mr. Alexander Ross, Vice-President, Institution of Civil Engineers ; Mr. Arthur R. Sage ; Mr. Percy W. Sankey, Assoc.M.Inst.C.E. ; Mr. A. Alban Scott, M.S.A. ; Mr. L. Serraillier ; Mr. C. W. Sharrock ; Mr. J. Osborne Smith, F.R.I.B.A. ; Mr. J. B. Travers Solly, Assoc.M.Inst.C.E. ; Mr. Alfred Stevens ; Mr. Percy M. Stewart ; Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A., F.S.I., etc. ; Mr. B. Taylor, M. Salford Eng.Assn. ; Mr. John M. Theobald, F.S.I., Q.S.A. ; Mr. Thomas P. Tinsley ; Mr. T. A. Watson, Assoc. M.Inst.C.E. ; Mr. E. P. Wells, J.P. ; Mr. H. D. Searles-Wood ; Mr. G. C. Workman, M.S.E. ; Mr. M. E. Yeatman, M.A., M.Inst.C.E., M.Am.Soc.C.E. ; Mr. Percy L. Young, A.M.I.Mech.E. ; and the Secretary to the Concrete Institute, Mr. H. Kempton Dyson.

GUESTS OF MEMBERS.

Mr. E. J. Allen ; Mr. Harold H. D. Anderson ; Mr. W. R. Barlow ; Mr. Percival Chubb ; Mr. M. W. Clark ; Mr. H. Clough ; Mr. C. H. Gage ; Mr. H. R. Goodrham ; Mr. H. J. Grace ; Mr. L. Griggs ; Mr. Harold Holloway ; Mr. B. van Homan ; Mr. W. R. Jaggard, A.R.I.B.A. ; Mr. R. Johnson ; Rev. Eyton Jones ; Mr. D. Mason ; Mr. G. J. McHaffie ; Mr. John Murray, F.R.I.B.A., Crown Surveyor ; Mr. J. Coulson Nicol, A.R.I.B.A. ; Mr. F. W. Pearce ; Mr. S. R. Potter ; Mr. Frederick Shingleton ; Mr. E. Simpson ; Mr. R. N. Sinclair, Chief Assistant Engineer Southampton Docks ; Mr. B. J. Skidmore ; Mr. H. E. Tinslay ; Mr. John Todd, District Surveyor for City of London East ; Mr. F. W. Troup, A.R.I.B.A. ; Mr. J. A. White ; and three others.

MR. ELLIS MARSLAND, representing the District Surveyors' Association, in proposing the toast of "The Concrete Institute," said :—I have been honoured in being entrusted to submit to you the toast of the evening, that of "The Concrete Institute." I have watched the progress of the Institute from its inception in 1908, I think, to the present time with very great interest, and my keynote to-night is one of congratulation, first of all on account of its membership, secondly on account of its objects, and thirdly on account of its work. Firstly, on account of its membership, it is a matter, I think, sir, of considerable congratulation to this Institute that in the short space of time of six years its membership exceeds one thousand members, and contains the names of most of those interested in the matter of reinforced concrete, both from a theoretical and a constructive point of view, and to my mind, sir, shows that there was a very great necessity for an Institute of this kind. In the early days of reinforced concrete constructions one was left to the tender mercies of various patentees and rival firms, who magnified their own systems to their rivals' disadvantage. But this Institute has changed all that. This Institute has evolved order out of chaos, has standardized and regulated the whole system of construction so as to make it practical and workable. It has assisted and taken a

very large part in perfecting the by-laws to be made under the 1909 Act in reference to reinforced concrete construction, and these by-laws, although they are not yet promulgated, will, I think, be the leading set of by-laws for the whole of the country when they are once past the authorities. Its members have read many useful and instructive papers, and some have written useful manuals on the subject. A very important work, I think, with regard to this Institute is still on hand, and that is inquiries into the failures which occasionally occur in reinforced concrete constructions and ascertaining the cause. Of course, in every new form of construction there is always a certain amount of prejudice in regard to it. I do not mean to imply that with regard to reinforced concrete constructions that there are more failures in this form of construction than in any other ; in fact, my own opinion is that there are less. Still, at the same time, the public want reassuring on this point, and although there have been some failures, I think it is the duty of the Institute to go into the question and to ascertain the cause of these failures and to bring about a remedy. I have to couple with this toast the name of your worthy President, Professor Adams. I have worked with him in another place for a great many years, and if he is as useful as he was to us I can assure you that you have an excellent President.

THE PRESIDENT, in responding, said :—I feel it should have been your late President (Mr. Wells) who ought to have had the honour of responding, especially when one remembers the great services he has rendered to this Institute. I do not think he has been absent from a single meeting during the two years—at any rate, whenever I have been there he has been there. However, as it is the custom in this Institute for the incoming President to undertake this duty as the first-fruits of office, I willingly reply to the toast. I thank Mr. Marsland for the kind way he has proposed it, and you, gentlemen, for the hearty manner in which you have received it. As you all know, the Concrete Institute was founded in 1908, and has achieved a notable result in obtaining over

one thousand members in that short time. That, I think, is evidence enough that it has a distinct mission to perform. It has many interests, and by means of its committees has already done a considerable amount of useful and practical work. It came in on a flood-tide, and its immediate success was almost phenomenal. The meetings were crowded, and copies of the papers were eagerly sought after ; then, unfortunately, there was an ebb ; the attendance fell off because it was found that in concrete alone there was not sufficient scope. The Council in considering the matter came to the conclusion that some development was wanted, and I think they wisely decided to admit papers on structural steelwork, where no concrete was employed or only employed in a secondary manner. The result was that the interest in the meetings was revived and the attendance went up again with a bound. The Council then fully considered the whole position and came to the conclusion that some amplification of the title was desirable, and added a sub-title, "An Institution for Structural Engineers, Architects, etc." Although this alteration was welcomed by many, it did not give universal satisfaction. The cement makers, chemists, contractors felt that they were being left out in the cold. If I were in their place, I should have felt the same. We already know a great deal about concrete, but there is a great deal more to learn, and in order that we may make further progress it is necessary that the various interests should combine together into one harmonious whole. Structural engineering is the one term that best expresses the variety of work of which this Institute takes cognizance, but that does not mean structural steelwork alone. It includes all varieties of buildings in all kinds of materials where stability is the chief aim. It excludes architecture, town-planning, railway and tramway work, sewerage work, waterworks, and mechanical engineering of all kinds. It appears to me that there is at the present time on large buildings room for the collaboration of the architect and the structural engineer. The architect has a considerable amount of work in considering the planning and ornamentation of the buildings, and he may be to some extent relieved by the engineer being respon-

sible for the stability. Broadly speaking, the architect is generally an artist; the structural engineer should be a scientific man and a mathematician, and that would roughly indicate the nature of the work that each would undertake. If, however, the architect is also a structural engineer, he would combine both duties; but it is not given to every man to combine these two branches of work. Then we have the burning question of the position of the so-called specialist; but why this term should be appropriated by the contractors' experts I fail to see. Any man is a specialist who devotes himself to a special class of work or makes himself acquainted with all that is known about it. Specialist firms undertaking work are simply contractors, but their experts are structural engineers of the highest class, and I consider them as the mainstay of the Concrete Institute. I certainly do not look upon them as tradesmen, although I have been told that they are tradesmen because they are engaged by contracting firms.

For some time past the Council has been preparing a scheme for examinations, and I hope by the autumn it will be in full working order. A complete copy of the syllabus is appended to the annual report of the Council, so that you may see what is included; but other subjects may be added from time to time. The intention is that the Graduates and Associate-Members should be able to obtain certificates testifying to their knowledge of the various branches that come under the domain of the structural engineer and architect. At present the examinations will be entirely voluntary, but there is no doubt that the time may come when the question of making them compulsory will be considered. The policy of holding examinations has been questioned in some quarters, but an examination certificate is the best form of testimonial, and with so large a proportion of young men among our members the opportunity of obtaining a certificate, I think, should be of great assistance. Having been unanimously elected by your Council to the high office of President, I look forward confidently to the support of the members of all classes. We have a large Council and many Vice-Presidents, and I do hope that we

shall see them in larger numbers at our ordinary meetings. We have also a Secretary who is not only a good business man but an expert in reinforced concrete. His personal help upon the committees has been of great value. In conclusion, I may say that the Council would gladly welcome any suggestion from any source that would tend to put the Concrete Institute upon a firmer basis and make it more useful to the members generally. The Council is frequently considering different matters that do not always come to fruition, but it is always open to receive suggestions.

MR. H. D. SEARLES-WOOD, F.R.I.B.A. (Vice-President of the Concrete Institute), in proposing the toast of "The Visitors," said :—We have a great many distinguished visitors here to-night, but they are all so well known that they do not need any introduction from me ; besides, their merits are all set out upon this table list. On behalf of the Concrete Institute I wish to thank these gentlemen for the honour they have done the Institute by attending the dinner. May I take it that the societies which these gentlemen represent show by their presence to-night that they are in concord with the Concrete Institute, the very Benjamin among the brethren in respect of age, but a lusty youth which they welcome into their family circle. I have to couple with this toast the name of Sir John Cockburn, ex-Premier for South Australia and sometime Agent-General for the same colony, now and always ready and willing to help any good cause forward. He, though a medical man by calling, has found time in his busy career to learn all about reinforced concrete, and he knows more about that subject than most of us here to-night.

SIR JOHN COCKBURN, K.C.M.G., M.D., late Premier of South Australia, in responding, said that, having recovered from a little nervousness which an interposed song had produced, he was very glad to have the opportunity and honour of replying for "The Visitors." Mr. Searles-Wood had already given away the show, because he must confess that though a professional man in other respects in concrete work he was a quack. (Laughter. He began his practice

of quackery in reinforced concrete in the year 1880, and he was looking forward to the Whitsuntide holidays to putting in the largest job in reinforced concrete that he had ever undertaken—an unsupported section of roof, 20 ft. by 12. He was very proud to be amongst the guests, because he recognized that this was a live society. There was not that dull monotony and uniformity of opinion which was death to all institutions. (Hear, hear.) They had the thrust of divergent views and the tie of association binding them all together. In fact, they had the essence of reinforced concrete in the society. He thought it was possible to reconcile the architect and the engineer, for he had noticed in the architecture and arts of the ages that what at one time was a structural necessity became afterwards a feature of ornament. The buttresses in the old cathedrals which were necessary to counteract the thrust of vaulted roofs were still continued in chapels where there was nothing but galvanized iron on the top. (Laughter.) What was often regarded as a disfigurement in one age became an element of beauty in the succeeding age. There was not the slightest doubt that in the development of concrete work lay both the architecture and engineering of the future. There was hardly a building being put up without some sort of concrete work in connection with it. Sanitary work owed everything to concrete. (Hear, hear.) Without concrete water conservation on modern lines would be impossible. Every building should be capped with concrete. Take the miserable position of their present roofs, whether they were slate or tile; they were neither frostproof nor windproof nor rainproof. He had the misfortune to live in a partly tiled edifice, and he said, "If anything heavier than a sparrow alights on those tiles they smash. (Laughter.) I keep peacocks. I regard them as artistically necessary. (Laughter.) If my peacocks alight on the roof, down comes a tile, and the masons have to go to the roof, and in their passage up and down they break half a dozen more. (Laughter.) Our modern residences, whether they are cottages or mansions, are like incomplete boxes. A box standing with its four open sides has nothing like the stability that it has when

you put the lid on. That makes it firm. No roof now goes on my house except the concrete roof." He had not found any difficulty with waterproofing concrete. Compact, well-mixed concrete ought to be watertight. There were several preparations which remedied imperfect concrete. He had not found any damp coming through his concrete roofs at all except in one spot where there was a crack, which he cured by what might be regarded as a quack remedy. (Laughter.) As a concrete quack he had an affinity with such. He had found what appeared to be a form of finely divided iron quite effectual in filling up cracks. The whole of the district in which he lived was full of the ruins of attempted concrete ponds. In his experience the best way to make a concrete pond watertight was to admit the water at the close of each day's work. For more than thirty years he had been in the habit of mixing silicate of soda with the upper layer of concrete to increase density. There is only one member of the community, continued Sir John, who has to regret the great progress which this Institute has made and which lies before it in the future. There is not the slightest doubt that reinforced concrete will greadly add to the woes and difficulties of the housebreaker. My experience of reinforced concrete is that nothing but dynamite will settle the question. (Laughter.) I do congratulate the Institute on its work and thank them for having done me the honour of inviting me here to-night. We feel that in the work you are doing you are not only adding distinction to the various professions of those who are members of the Institute, but that you are also engaged in a great imperial work. (Cheers.) Without reinforced concrete the great engineering feats in Australia and in the British possessions would have been impossible. Concrete construction is still in its infancy, and it has a glorious future to look forward to.

Mr. E. FIANDER ETCHELLS, F.Phys.Soc., proposed the health of the Chairman. His remarks were extremely humorous. He observed at the outset that during the dinner he had noticed one preoccupied diner eating asparagus at the wrong end. (Laughter.) On his attention being drawn to the fact, he replied,

"I beg your pardon, but it tastes like spring onions." (Laughter.) He made a suggestion to the management that the asparagus would be much more convenient for eating if it were reinforced. He stated that he had recently read a Life of Professor Adams, and it seemed that the people of the Professor's family had been engineers right from the very first person of that name. (Laughter.) While the senior members of this Institute were at school Professor Adams was busy getting his teeth into reinforced concrete. When he served his apprenticeship there was no wireless telegraphy to send out lying messages, there was no submarine to sink its living burden, no motor-cars to endanger our lives, no telephones to be a nuisance, no stuffy tubes at twopence a time, and no aeroplanes to crash to earth. These changes have come about during the engineering lifetime of our President. He has not been responsible for these. He has been responsible for more enduring things. To give you a glimpse into the conditions when Professor Adams put up his first building I have some extracts from the "Life of Charles Matthews," a man whom Nature intended to be an actor, but whose parents decreed that he should start life as an architect. (Laughter.) He went to Rome and Florence and saw beautiful buildings, and his soul was filled with visions of beauty. He came to London and was made District Surveyor for Bow and Bethnal Green. (Laughter.) However, he was not deterred by trifles. (Laughter.)

"The only touch of joy I had [he says] was on the discovery of a locality rejoicing in the name of Cut-throat Lane, and in no other place could I make up my mind to fix my office. 'District Surveyor, Cut-throat Lane,' was something to have on one's card, and gave a spice of romance to the affair. The emolument arising from the appointment was startling, and about £40 per annum compensated me for my agreeable labours—that is, would have done so had I received it, but there was the difficulty. It consisted of fees—fees to be collected by myself in person, and a pretty time I had of it. At one house I knocked humbly after considerable hesitation. The door was opened cautiously with the chain up, and a stout suspicious-looking dame, in a pair of nankeen stays, asked me if I came 'arter the taxes or summat?' 'No, madam,' I said deferentially, 'I am the district surveyor from Cut-throat Lane, and I have called for——' 'Oh, bother!' said the lady; 'summons me if you like—I'm not going to be humbugged by you.' Another defaulter kept an oilcloth warehouse in Whitechapel. It was some time before I could muster up courage to enter, as there were several customers assembled. However, I ventured in, and was met by an appeal that was irresistible. 'What,'

said the master of the establishment, 'you a gentleman, come to a poor man like me, and ask me for such a paltry sum as that? You ought to be ashamed of yourself!' Turning with indignation to his customers—'What do you think of this? Here's a gentleman who——' I did not stop to hear the rest, but made my exit at once. I was not curious to hear the opinions of his friends, and thought myself lucky to escape being tossed in a blanket. Shades of Vitruvius! was this architecture?"

"Now," Mr. Etchells went on, "speaking of the district surveyors reminds me of the Building Act, and I may remind you that of the suggestions made by this Institute to the authorities in respect of the draft regulations no less than about 90 per cent. have found favour in high places."

THE CHAIRMAN briefly responded.

FIFTY-FIRST ORDINARY GENERAL MEETING

THURSDAY, NOVEMBER 19, 1914

THE FIFTY-FIRST ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, S.W., on Thursday, November 19, 1914, at 7.30 p.m.,

THE PRESIDENT (PROFESSOR HENRY ADAMS), M.Inst.C.E., M.I.Mech.E., M.S.A., F.S.I., F.R.San.I., etc., in the Chair.

The following elections took place :—

MEMBERS.

HERBERT JOHN DAVEY, Inspecting and Testing Engineer, London.

GERALD ECKETT DUNNAGE, Architect and Surveyor, London.

HENRY JAMES-CARRINGTON, M.I.Mech.E., M.Met. Soc., Consulting Engineer and Works Architect, Birmingham.

ROBERT PEEL MEERS, B.A. (Cantab.), Stud.Inst. C.E., Bombay.

JOHN MILLAR, Lecturer in Building Construction, Surveying, etc., London.

L. REMY, Architect and Builder, Rio de Janiero.

CHARLES ADOLPHUS STEVEN-JENNINGS, M.Soc. Arch.S.A., M. Surveyors' Inst., S.A., J.P., Architect and Surveyor, E. London, S.A.

WILLIAM WELLS, M.Inst. Water Engineers, Stud. Inst.C.E., Civil and Waterworks Engineer, Antofagasta, S.Am.

ASSOCIATE-MEMBERS.

HALSTEAD BEST, M.R.San.I., Stud.Soc.Arch., Structural Engineer and Architect, Dublin.

DHARMAI KUMAR BOSE, Engineer, London.

FREDERICK LIONEL BROWN, Civil and Mechanical Assistant and Draughtsman, London.

ERNEST A. H. DAWE, Assoc.M.Inst.C.E., Engineer, Sydney, Australia.

HAROLD CHARLES DIXON, Draughtsman, Stratford.

HERMAN PETER H. LIND, M.Soc.Danish C. Engineers, B.Sc. Civil Engineering, Designing Engineer, London.

JAMES JOHN LANE OAKEY, M. Clerk of Works Association, Lecturer and Teacher in Building Construction, Cheltenham.

BLAND THOMAS WILLIAM OWEN, Assistant Civil Engineer, Dunedin, N.Z.

ASSOCIATES.

JOHN HENRY DUNN, Builder and Contractor, Brazil.

SAMUEL GLUCKSTEIN JOSEPH, Builder, London.

GEORGE HENRY WALTER EDWIN KENT, Inspector of Works, Cape Colony.

JAMES TYLER, Clerk of Works, Auckland, N.Z.

STUDENT.

FREDERICK JAMES T. HOLLAND, Structural Draughtsman, London. (Has been admitted as a student of the Institute by the Council.)

THE PRESIDENT read his address as follows :—

GENTLEMEN,—In opening the present session with a few personal remarks it is my first duty to express my appreciation of the great honour which has been conferred upon me by your Council, who by their unanimous vote have elected me to the presidential chair for the next two years. This is in accordance with the custom which has obtained in this Institute since its foundation, but in these democratic days I am doubtful whether such a course commends itself to the membership, or is consonant with the trend of public opinion. I would rather see the nomination of President and Vice-Presidents submitted to the vote of the general

body of the members. This matter is under the consideration of the Council, and I hope that the various alterations in the Articles of Association which are proposed will include some means by which the members all over the world can take part in the elections, not only of the chief officers but also of the Council generally.

The Institute is, I am happy to say, in a very flourishing condition. There are at present on the roll 16 honorary members, 933 ordinary members, 27 associate members, 6 associates, 51 students, and 5 special subscribers. Although the numbers are for the present satisfactory, it must be remembered that there is a constant outflow due to deaths and resignations, and that merely to maintain the *status quo* about fifty new members are required each year. With a young and progressive Institute like this there ought to be no difficulty whatever in doubling this number, and as the influence of the Institute, and its capability for benefiting its members, increases with every addition to the membership, it is to be hoped that efforts will not be slackened until the two thousand is reached.

The unfortunate war now in progress has already had the effect of checking building enterprise, and it may be some considerable time before a recovery takes place, but beyond this, the call of the nation for volunteers to join our Army and take part in the defence of our coasts must have a depressing effect upon the membership roll, and it will be no cause for wonder if there should be a temporary falling off.

Our financial position is satisfactory, although last year our expenses exceeded our income by a small amount. This was due to various special causes, chiefly the printing charges, which were in excess of the usual amount, owing to the number of tables and diagrams in the TRANSACTIONS. The annual subscription, notwithstanding it having been increased, is still below the average of other Institutions, but the Council consider that it will be sufficient to carry on the ordinary work and leave a small margin for developments. When our funds increase there are many useful directions in which we could employ the surplus.

A library is in course of formation and a catalogue

will shortly be issued. The new methods of construction are developing so rapidly that a constant study of the literature of the subject is necessary, and the Council wish to give every facility to the younger members to improve themselves at a minimum cost. Naturally some of these younger members are too diffident to join in the discussion at the ordinary General Meetings, and the Council have therefore inaugurated a series of informal monthly meetings for the junior members only, at which a member of the Council takes the chair and friendly discussions ensue upon a subject announced beforehand, or upon a short introductory lecture given by one of the members.

The Report of the Council presented at the Annual General Meeting held in May last was so complete that I need not take up your time with recounting the details of the papers, visits, committee work, and so on, which often help to make out a presidential address. I would only say that as far as possible the activity which characterized the last year's work will be fully maintained during the coming year ; but the war must affect our Institute and the work of its members in one way or another, so that, without endeavouring to foresee the ultimate results, I would say that for the present our motto is, "Business as usual."

The extreme importance of the official "Regulations for the Use of Reinforced Concrete" in London, in the preparation of which this Institute, with other bodies, was consulted, has caused the Council to devote a considerable amount of time and anxious thought to the discussion of the various propositions during the last five years. While they desired to place no difficulty in the way of sufficiently safeguarding the public in the use of this method of construction, they were, on the other hand, in duty bound to their members to see that no unnecessary restrictions were placed upon it. For the first time in history the authorities have suggested an alteration in the laws of mechanics in order to provide against bad workmanship, and I cannot help thinking that this is a mistaken method of securing safety, and in any case the formulæ should have been relegated to an Appendix. What would be thought if, in the Steel-frame Building Regulations, incorrect bending moment formulæ were given by which the

girders were to be calculated to compensate for insufficient riveting to resist shear? In that document results only are stipulated for, the stresses being assumed to be calculated by the ordinary laws of mechanics.

In M. Béhar's paper, on "The True Bending Moments of Beams with Various Degrees of Fixity," the matter was summed up as follows: "It appears obvious to me that an engineer or architect who may be called upon to apply regulations concerning reinforced concrete should have sufficient knowledge of the laws of mechanics to be able to deal with the various problems which he may have to study; and to oblige him to follow certain formulæ would practically imply that he is incapable of exercising proper control of any scheme or problem in reinforced concrete which he may have to consider."

You are probably well aware that the actual distribution of stress in a complete structure can only be approximately estimated, and that reliable data for constructional work can only be slowly gathered by extended experiments and minute measurements of the induced strains, which take both time and money, but until we have more precise knowledge I maintain that if we adopt the accepted laws of mechanics we shall not go far astray. I must not say any more about these Regulations at present, as they are still *sub judice*, but I do hope that when they appear they will be in a form to command respect as well as compliance. I would, however, just add that in the last proof that has been received, out of 87 suggestions made by this Institute, 65 have been adopted, and there is some ground for hoping that the final revision will include a good proportion of the outstanding balance; at any rate, we are assured that facilities will be given for revision from time to time after the Regulations have been tested in practice.

If the Concrete Institute has emphasized one thing more than another in connection with the materials the members are interested in, it is the necessity for the best possible workmanship and the utmost care in supervision. The majority of the failures that have occurred have been due to improper materials or careless workmanship. Part of the secret of good work is good pay for it. Where labour is cheap the output is

poor both in quality and quantity. We have a good illustration of this in the employment of natives in India :—

> Oh, life is cheap in the golden East,
>   And labour even cheaper ;
> *But the seed for which you have paid the least*
>   *Brings least at last to the reaper.*

In my remarks upon the subject at the International Congress of Architects eight years ago I paraphrased the old saying that "nothing succeeds like success" by "nothing succeeds like failure," because I believe that a proper study of failures will do more than anything else to ensure success. In 1912 the Concrete Institute formed an "Investigation Committee" for the purpose of studying failures as they occurred, but the natural desire of all parties concerned to hush up the failures has prevented the Committee so far from doing much useful work, unless the fact of its existence acts as a bogy to frighten the careless operative. It may be well to remind you that a schedule has been drawn up for recording the details of such failures as are brought to the notice of any member, and all such schedules are treated as confidential by the Committee unless permission is given by the parties concerned to make public use of them. It is the misfortune of engineers that their successful works, unless of exceptional magnitude, are without interest to the man in the street, while their unsuccessful works attract more attention than is desired. Architects are not quite so badly off, because their works appeal more to the eye of the general public, and if anything does go wrong they meet with some sympathy. The bond of association which renders it possible for engineers and architects to meet freely within the walls of our Institute lies undoubtedly in the fact that the subjects discussed have similar interest for both. There is a sort of neutral zone in reinforced concrete construction where the engineer and architect are equally at home, and this alone renders our Institute unique in its membership.

The scientific principles of reinforced concrete and structural mechanics are now so well known that the failures due to ignorance on the part of the designer are conspicuous by their absence, but frequently there

are failures reported which are due to the formwork being badly designed and fixed, or removed too early. Too often these matters are left to the foreman carpenter, and however conscientious he may be his goodwill is no substitute for experience. *Festina lente* ("Make haste slowly") is, I think, an exceptionally good motto for us and our work. Near Auburn, California, a reinforced concrete bridge in three spans was under construction towards the end of last year. The work was being rushed day and night by the contractors in order that it might be finished before the river rose, and on the night of November 4th the formwork under one end span collapsed, allowing the whole mass to fall, killing some of the workmen and injuring several others.

In engineering works, such as reservoir dams, other causes of failure have been found, one of the most remarkable of which is the action of alkali on concrete. Where junctions occur between successive portions of the work a leakage is most likely to take place, and if any alkali exists in the earth resting against the wall which the leakage can reach, a very rapid disintegration of the concrete is found to take place.

A curious objection has been taken to building in reinforced concrete, which is, that it is so difficult to pull down. It is frequently said that it costs more to destroy a building than it does to put it up. It must be admitted that this method of building does not lend itself readily to alterations; and although I have not personally come across a case where this has been a serious matter, there is a recent instance in Germany where some concrete vaults which had to be removed cost twice as much to destroy as they had cost to build. In some other cases it has been found that only by recourse to blasting operations has it been possible to break up the work in this material, but the imperishable character of the structure and the small cost of upkeep are really very great points in its favour. Their inherent strength and the nature of their stability render these buildings peculiarly suitable to withstand vibration, and even earthquake, whereas ordinary construction is only stable while at rest, and the slightest earth tremors may have a disastrous effect.

Brickwork is one of the best fire-resisting materials, but a building cannot be erected in brickwork alone,

and considerable loss of life has occurred in fires from the walls falling as they ceased to receive the support of the floors and roofs. Reinforced concrete, on the other hand, can be used not only for the walls but for the floors and roofs also, and the steelwork, which is often the cause of the greatest danger in a brick building, is so protected by the concrete that the fire has practically no effect upon it. Limestone must not be used in the aggregate, as that burns to pure lime in a fire and swells and disintegrates when water comes upon it. Coke breeze as an aggregate was found to stand fire well in the experiments made by the British Fire Prevention Committee, and for that reason has been largely used by architects, but coke breeze has sometimes a deleterious effect upon the steel reinforcement, and is liable to swell in damp situations or if water reaches it. An indication of the value of reinforced concrete is given in the fact that a lower premium is asked by the Fire Insurance Committee for buildings in this material providing the floors are not less than 5 inches thick and the walls not less than 6 inches thick.

It will be interesting to look back upon some of the troubles that faced the early workers in reinforced concrete and to see how such troubles have been surmounted. One of the most frequent difficulties found in the early days of reinforced concrete building was the occurrence of cracks in the finished work. Some of these were undoubtedly due to over-limed cement and to coarse grinding, but the improvement effected by the publication of the Standard Specification of Portland Cement and the endeavour of manufacturers to keep up with it, and even to surpass it, have virtually eliminated this cause of trouble.

A still frequent cause of cracking is shrinkage upon the drying out of the moisture, but a remedy for this has been found in what is called "temperature reinforcement," which consists of the insertion of light rods, say $\frac{1}{4}$-in. diameter, and not more than 12 in. apart in the opposite direction to the main reinforcement and near the surface, whereby the shrinkage, being confined within these limited areas, is so slight as to be invisible. Other causes are sudden changes of thickness, changes in the composition of the concrete, local currents of

air upon the concrete while setting, insufficient reinforcement allowing deformation to occur, etc. Where the new surface is exposed to the sun the changes of temperature are apt to cause "crazing," particularly where the surface skin is richer in cement than the body of the work. The best remedy for this is perhaps to brush the surface with hydrochloric acid and then wash it with clear water.

Another cause of early difficulty was the uniformity in size of the larger aggregate. When the maximum of $1\frac{1}{2}$ in. was reduced to 1 in. it was thought that this would be effective in producing concrete capable of being packed closely round the reinforcement, but tests showed that it was necessary to grade the aggregate from $\frac{1}{4}$ in. to $\frac{3}{4}$ in. to make sure of obtaining a solid concrete without voids and in perfect contact with the steel. In the same way the sand was found to produce better work when it was not of uniform size but graded from $\frac{1}{50}$ in. to $\frac{3}{16}$ in. As the cement has to envelop every particle of the aggregate, and cement one to the other, the grading involves the use of a smaller quantity of cement as the voids are small, and also reduces the waste of aggregate as nothing but dust has to be thrown away.

Much valuable work has been practically lost because of the difficulty of co-ordinating experiments made with different materials under varying circumstances. An important suggestion has been made by one of our members, Mr. John A. Davenport, that for some time to come all experiments on reinforced concrete beams, columns, and similar structural members should be made on a standard steel and a standard concrete. By "standard steel" is meant one having a particular ultimate strength, yield point, elongation, and contraction; these could vary between limits, but the variation should be kept small. By "standard concrete" is meant one made of one particular coarse material, one particular sand, and one particular cement, always mixed in the same proportion with the same amount of water. The coarse material should always have the same sizes and grading, and the sand should likewise have definite sizes and grading which must always be the same. We should then be able to decide definitely many of the points which were the subject

of lengthy discussions during the preparation of the Local Government Board Regulations—for example, what constituted fixed ends to beams, what width of floor slab should be taken as the flange of a T-beam, what was the actual economy of double reinforcement, how far a concentrated load might be considered to have spread by the time it reached the reinforcement, the comparative value of vertical and inclined shear stirrups, adhesion between the concrete and the reinforcement, the economical value of deformed bars, etc.

A Joint Committee of the Concrete Institute and the Quantity Surveyors' Association have prepared a synopsis of a "Standard System of Measurement for Reinforced Concrete Work" which will greatly facilitate uniformity in the preparation of bills of quantities and reduce the risk of errors of misunderstanding and consequent disputes. The story of a remarkable blunder was related in the Chancery Division of the Law Courts on January 30th last, when the Commissioner of Works was allowed rectification of a contract with Messrs. W. King & Co., contractors. The Solicitor-General explained that the contract was for the construction of the Western District Post Office building in Wimpole Street. The contractors scheduled reinforced concrete at so much per cubic yard, and when the schedules were sent to be typewritten "yard" was altered to "feet," the effect being to alter the price from £20,000 to more than £133,000.

The Council have in hand a standard specification for reinforced concrete which is nearing completion, and if more uniformity in drafting specifications for such work can thereby be obtained there will be more certainty in the tendering and a greater absence of sporting items. The absence of immoderate or harsh clauses should lead not only to better work but also to cheaper construction. In publishing a model specification, however, and in widely circulating the information contained in the papers read at the meetings there is some risk of encouraging incompetent men to attempt to prepare designs and carry through construction in reinforced concrete; but although such works would be likely to be small and insignificant, it cannot be pointed out too emphatically that success

can only be obtained by employing men who have specialized in this class of work. Three years ago in Boston, U.S.A., the corner of a large building collapsed owing to improper foundations being put in for a column supporting a great weight, and the contractor was killed. He was acting as architect as well as builder, although a few years previously he was only a teamster. Was this an accident or a just retribution? Nature has no mercy, and if her laws are broken the penalty is certain. At the Sixth Congress of the International Association for Testing Materials, held at New York in 1912, the question of universal standard specifications was mooted, but there seem to be insuperable difficulties in the way at present. It was suggested that when each country got its own perfect specification it would be time to compare them and arrive at a single result, but I am afraid we are a very long way yet from that happy end.

I think it may be useful for us to consider for a few minutes what is the usual method followed at present for obtaining the design for the structural parts of a reinforced concrete building and whether that method conduces to economy. The routine in most cases is for the architect to prepare an outline drawing showing the plan of each floor, with the cross walls, openings for doors and windows, and positions of stairways, lifts, etc., marking on it the loads to be carried and the height of each floor. He then obtains designs and estimates from a certain number of firms who devote themselves to this class of work but are each identified with a particular detail of construction, and selecting the offer which he considers the most suitable and economical, completes his own designs, and puts them before his client for approval. Now the question is, Does this method lead to economy and security? The designs so obtained, being competitive, are naturally based upon the minimum requirements of safety, and leave no margin for accidental errors or faulty material or workmanship, and some of the failures may possibly be due to this cause. I do not intend this as any reflection upon the skill or honesty of the specialists who at present prepare the majority of the designs, but it is not in human nature to provide more than the bare necessity of the case demands when

the success of the tender depends upon the cost being kept down to a minimum. The Institution of Civil Engineers has recently issued a circular deprecating the preparation of designs in competition. As a consulting engineer, I may perhaps be biased, but I am of opinion that collaboration between the architect and the structural engineer is the preferable course, so that a single design may be prepared upon which contractors may tender upon a common basis. Against this it is often said that engineers' designs are expensive, and that there is no advantage in calling in an independent expert to prepare the design ; but I think it will generally be found that the additional cost is due solely to a larger amount of reinforcement than appears in competitive designs, with the result that the building gains in greater stability.

There is another aspect of the case, and that is that when several designs are submitted the cost of the unaccepted designs has to be met somehow. It increases what we may call the establishment charges, which must be met by a percentage added to the actual cost of all work, and the only advantage appears to be that with many men working upon the same scheme some one among the many may evolve a better result than one man alone would be likely to do.

The Local Government Board are responsible for retarding the progress of reinforced concrete to a very great extent. Many structures that would have been put up in that material have not even got so far as the design, because of the known opposition of the Board and the heavy expense thrown upon the promoters by reason of the short period for which a loan would be granted. They have fixed no actual periods for loans for the various classes of work, as each case is considered on its merits, having regard to the purpose of the work and its position, but they vary usually from ten years where the work is in contact with water to thirty years in the floors of a building.

If there were any reasonable doubt as to the probable duration of reinforced concrete structures in general, the Council of the Concrete Institute would be the first to know it, but they have the evidence of their own personal investigations that it exceeds fifty years,

and they do not hesitate to advise its use where that is not hampered by the consideration of a loan through the Local Government Board. When the Eddystone Lighthouse was pulled down in 1884 a bundle of iron rods which had been accidentally left in the concrete foundation in 1757 (127 years) was found to be in perfect condition, and one would have supposed that was in a sufficiently exposed situation to prove the safety against corrosion.

Within my knowledge the construction of a large reinforced concrete seaside pier was quite recently abandoned after completion of the designs because the Local Government Board would only allow ten years for the repayment of a loan for its construction. In another case the Local Government Board refused to allow more than ten years for the repayment of a loan of £10,000 for a reinforced concrete storage-tank. The tank was then re-designed in mass concrete to cost £13,750, for which the Board allowed twenty-five years for the loan repayment, making the annual charge £880, as against £1,233 which would have been required for the less costly tank on the ten years' basis. Had twenty-five years been granted in the first case, the annual charge would only have been £640. Fortunately, there are many cases that do not depend upon official mercy. A sea wall, 300 ft. long, in reinforced concrete has just been erected at Carnarvon by the Harbour Trust in extension of an existing masonry wall, to afford increased wharfage for oil-tank steamers and to reclaim about two acres of the foreshore. The existing heavy masonry structure, 20 ft. high from shore to coping, rested on a foundation 6 ft. below shore level, and it was originally intended to construct the extension in a similar manner. There were many ingenious features in the design carried out, but the point I want to make is that the cost of this work in reinforced concrete was about £2,000, while the lowest tender for a masonry wall was £4,600. The corrosion of the reinforcing rods when the concrete is in contact with water, fresh or salt, continuously or intermittently, is only possible under exceptional circumstances.

A novel application of reinforced concrete has recently been made in the construction of a floating

wharf in Sydney Harbour for berthing steamers on each side. The pontoon is 100 ft. long, 43 ft. wide at one end and 68 ft. at the other, 7 ft. 9 in. deep, with a 32-in. freeboard under full load. There are forty-eight watertight compartments, and the total dead weight is 650 tons. The live load allowed for is 150 tons. Apparently no fear of sea-water acting prejudicially upon the reinforcement troubles the minds of the promoters in this case.

So much has been said lately about structural engineering and so many persons seem to be in doubt as to what is meant by it that I should like to devote a short time to its consideration. It is a great mistake to suppose that it consists of steel-framed buildings only ; that is merely one branch of a very large subject. Reinforced concrete is another branch—a very large one, and possibly the main stem, at least for us. These two forms of construction will in the future, so far as we can foresee, constitute the chief work of the structural engineer, but buildings in other materials—wood, stone, and brick—come under the head of structural engineering when the chief object is stability. Bridges, towers, jetties, retaining walls, roofs, tanks, reservoirs, dams, and many other structures would come under this classification, and it is to enable us to deal with all these in our papers and discussions that the Council have endeavoured to broaden the scope of the Concrete Institute. Bridges alone should be a fertile subject. So many examples have now been completed in reinforced concrete that we can form a fair idea of the value of that mode of construction. It has a distinct advantage over brickwork and masonry in being able to resist tensile stresses and changes of compression and tension due to rolling loads.

No one type is predominant, but the arch enters into most of them. An arch is not only a pleasing form of construction, it is based on sound scientific principles and is an economical disposition of material. I have considerable doubt whether it is desirable to construct hinged arches in this material, whether two-hinged or three-hinged. It appears to me that it is almost equivalent to using simply supported girders when you have the opportunity to make them con-

tinuous. There is, of course, the advantage that the line of thrust must pass through the hinges, but there is no difficulty in so proportioning an ordinary fixed arch that the line of thrust nowhere unduly leaves the thickness provided. It may be said that no one knows the exact course of the line of thrust, but we know that Nature takes the course of resisting a load with the minimum of stress, and we shall not be far wrong if we take the line of thrust so that the extreme stresses on the opposite haunches are as nearly equal as possible. The favourite method of construction seems to be a flat or cambered floor with cross and main beams and pillars from the main beams to the arch itself, the arch being a continuous sheet or ribbed sheet ; when the latter the pillars are braced transversely and the arch is stiffened by a repetition of main and cross beams similar to those under the bridge floor. In this work, as in all arches, it is necessary to provide rigid abutments, but reinforced concrete will withstand a shifting abutment better than any other material.

The progress made in arched bridge-building in reinforced concrete is remarkable for a comparatively new material. Among the larger examples we have 233-ft. span in the Walnut Lane Bridge, Philadelphia ; 320 ft. at Grafton, New Zealand ; 328 ft. over the Tiber at Rome ; 330 ft. at Largweiz, in Switzerland—all within 100 ft. difference. Now we have the proposed Spuyten-Duyvil Creek Bridge at New York, where a span of 703 ft. is proposed, or more than double.

The general movement that has taken place throughout the kingdom to improve the high-roads has not gone so far as it might have done in the direction of improving the existing bridges. As reinforced concrete is such an admirable mode of construction for highway bridges, the Concrete Institute formed a joint committee with other bodies to standardize the loading to be provided for in bridges of different classes. They collected information from all parts of the country of the weight and dimensions of the actual traffic, and decided what maximum loads each class of bridge should be rated to carry. Their labours are nearly completed, and it is hoped that they will

be able to produce a single chart that will reduce the draughtsman's labour to a minimum.

Although steel, according to the late Sir Wm. Siemens, has been used for structural purposes from the year of the Great Exhibition (1851), the modern use of steel framework in the construction of high buildings dates practically from 1881 in America, and more recently in England. The extensive use of steel in all classes of buildings at the present time makes the study of steel-framed construction of the utmost importance ; but it is a very great mistake to imagine that any member of this Institute wishes to put reinforced concrete in the background because of the importance of steelwork. Our Institute was built up on reinforced concrete, and it should be the primary care of the Council to see that it does not suffer by reason of any extension of the scope to other modes of construction. As one instance of the importance of a study of steelwork, I may mention that the Equitable Life Assurance building of forty-one stories now being erected in New York will contain nearly 45 acres of floor area ; and 35,000 tons of steel, held together by a million rivets, will be involved in the construction, the total cost reaching £6,000,000.

But steel-framed buildings, although the chief branch of metalwork to interest us, is not the only one. We must know something of bridges in metal before we can properly make a comparison with those in reinforced concrete, and it is not the smaller ones only that we should study. A steel bridge now constructing in this country for erection across the Ganges is a mile long, in fifteen spans of about 350 ft. each, and will contain 30,000 tons of steel. This is a gigantic affair and beyond the probable professional outlook of many of us, but there are many intermediate structures between that and a rolled steel joist which we can study and discuss with advantage.

The Science Committee have for a long time past endeavoured to gather useful information upon the driving and supporting power of reinforced concrete piles, but so far they have not been able to collect sufficient material for a report. Records of the weight and fall of ram, number of blows, and distance driven are very numerous, but comparatively useless. Some-

times the records are accompanied by particulars of a test, when the pile sustained a given load without yielding, but whether it would have yielded with ten or twenty times the test load we have no means of knowing. What is required is a series of experiments with small piles, say 10 in. square, 20 ft. long, driven 15 ft., and then loaded until they sink farther. This would do for a beginning and would enable us to compare reinforced concrete piles with timber piles. I have a collection of about forty formulæ for the latter kind, all more or less *unreliable*, but none at all for the former. At Halifax, Nova Scotia, the largest reinforced concrete piles have been recently driven by the largest pile-driver in the world. The piles were 2 ft. square and 77 ft. long; the hammer, follower, and follower-guide weighed 28,000 lb., and the ram alone weighed 4,000 lb. With a mean effective steam pressure of 80 lb. per sq. inch in the cylinder of the engine operating the driver the hammer is rated to develop 3,916,000 foot-lb. per minute when striking eighty blows per minute. Twenty-five piles have been driven by this machine in a ten-hour day. But this record is already exceeded by some piles at Havana, and tenders have been called for a pier at San Francisco in which piles of 91 ft. long and 20 in. square are required to be constructed. They will weigh 15 tons each, and are specified to carry in addition to their own weight a load of 40 tons. The concrete will be $1 : 2\frac{1}{2} : 5$, and will be allowed forty-five days for setting before being driven.

Reinforced concrete has been the means of saving a considerable expenditure of money in the cost of foundations otherwise than by piling. By enabling the pressure to be spread over a considerable area, without much doubt it has in many cases enabled buildings to be safely erected upon soil that would have defied the old method of putting in foundations. Concrete rafts have been used for a long time, but until they were reinforced it was not uncommon to find the heavily loaded parts punching their way through the concrete. Perhaps the greatest innovation is possible in connection with bridge foundations, which are often very deep, 100 ft. or more below high-water

mark. In the case of the Hawkesbury Bridge, in New South Wales, the foundations went down to 162 ft. below high-water level.

Other matters might have claimed some of our attention to-night, but we have probably travelled over sufficient ground to exhaust your patience.

As I am speaking to an audience of experts, I am afraid I have told you nothing new, but it is important to emphasize well-known facts when they tend to produce sounder work and increase the credit that is due to the material in which we are all interested.

MR. E. P. WELLS, J.P. (Past President C.I.), in proposing a vote of thanks to the President, agreed with his remarks as to the formulæ for bending moments in the Regulations.

SIR HENRY TANNER, C.B., I.S.O., F.R.I.B.A. (Past President C.I.) :—I have very great pleasure in seconding the vote of thanks which Mr. Wells has proposed to the President for his interesting and informing address this evening. The only remarks which I wish to make on the paper are those concerning the practical parts of it—for instance, as to the methods of obtaining tenders. I cannot see any reason why reinforced concrete should not be dealt with in the same way as any other part of building construction—that is, by preparing a specification and quantities in the usual way and obtaining tenders upon the latter. I am quite convinced that to ask any one for designs and tenders is wrong. Such a course leads to confusion and trouble, and it is never known whether the best and at the same time the cheapest design is obtained, because much less material may be used, although it may ostensibly meet the obligation. I am hopeful that we may see that course altered in course of time. When there is a standard specification and a definite set of regulations, it will still be more easy to follow that order. If the Local Government Board confirm these Regulations for London, they will no doubt be used throughout the country, and the Local Government Board should reconsider the question of repayment for loans, because if people know that the terms would approximate to those

allowed in the case of ordinary construction reinforced concrete work is more likely to be adopted than it is at present. The Institute might take the matter up, and it ought to stand some chance of success. I have had very little experience in regard to pulling down reinforced concrete structures, but alterations have had to be made, and there has not been any trouble whatever ; but in the case of ordinary concrete considerable labour has been involved, and in some cases blasting has had to be resorted to.

Mr. E. FIANDER ETCHELLS, F.Phys.Soc., M. Math.A., A.M.I.Mech.E., Hon.A.R.I.B.A. (Member of Council C.I.) :—I should like to associate myself with the two Past Presidents in supporting the vote of thanks to the present President. I am well aware that one may not criticize a Presidential Address, but perhaps one may be permitted to express agreement, even if it be a limited or qualified agreement, in respect of those points which are admittedly highly controversial or contentious. The Presidential Address contains a statement to the effect that, for the first time in history, the authorities have suggested an alteration in the laws of Mechanics, in order to provide against bad workmanship. Perhaps the President will agree with me when I say that bad workmanship ought to be provided for and guarded against. Whether it were best to provide for it in the bending moment or in the matter of stresses is a point which cannot and ought not to be discussed this evening, but we can all agree that some provision must be made. I would remind you that in Regulations it may be necessary to provide for something more than the laws of Mechanics. It is necessary to give consideration to the laws of Finance. It may also be necessary to take into account the laws of human nature (which the professors call Psychology). These laws enter into every building job, every time and all the time. There are also rules of prudence to be considered. If the President will mark a copy of the Regulations showing which particular equations are believed to be contrary to the laws of Mechanics, these particular clauses could be brought to the special notice of those whose decision must be final. I find myself

whole-heartedly in agreement with Mr. Wells when he says that the laws of Mechanics, particularly those in respect of bending moments, should be the same as for all other materials. That has always been my own contention. Reinforced concrete is subject to ascertainable laws, and is not the miraculous embodiment of mystical genius.

THE PRESIDENT (PROFESSOR ADAMS) :—Mr. Wells, Sir Henry Tanner, Mr. Etchells, and gentlemen, I thank you very heartily for the kind way in which you have proposed and received this vote of thanks. If I were to make any remarks in reply to what has been said, it would look almost like opening a discussion, but I might perhaps elucidate the point about the Western District Post Office. The particular point of that was the yards being changed to feet, and a few members only may be aware that, in our discussions with the Quantity Surveyors' Association, that was the point which took the longest time to decide, whether we should have cubic yards or cubic feet for the measurement of reinforced concrete. We decided in favour of cubic feet, because, being used in much smaller masses than the old form of concrete without reinforcement, it was likely to be better understood and priced more at its proper rate than if we had left it in cubic yards. With regard to the other point, the bending moment formulæ, I ought perhaps to say this in justice to Mr. Etchells, who has been of very great assistance to us, that several modifications have been introduced during the passage of the Regulations through our meetings, and they are now in a very much better form than they were even when that portion of this Address was written.

FIFTY-SECOND ORDINARY GENERAL MEETING

THURSDAY, DECEMBER 3, 1914

THE FIFTY-SECOND ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, December 3, 1914, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I. Mech.E., F.S.I., M.S.A. (President), in the Chair.

MR. H. KEMPTON DYSON then read his paper as follows :—

SHEAR AND PROBLEMS ARISING THEREFROM.

SEEING that tangential stresses are developed in connection with every structural form, including those members that are subjected to direct tension and compression, it is quite impossible to treat of every aspect of my subject within the limits of a paper. The selection of certain aspects for consideration which is made herein may not include aspects which are of interest to others, but the discussion to follow affords opportunity for important deficiencies to be pointed out so that they may be repaired by further contributions.

At the outset terms must be clearly defined so as to avoid misunderstanding, and for that purpose reference must be preliminarily made to some elementary matters.

The transmission of forces between two bodies or two parts of the same body results in equal and opposite action and reaction and constitutes a stress. The intensity of stress at a surface, however, is what is usually called a stress and will be used here in that sense. Stress will, therefore, be calculated by the force per unit of area, and in the case of variable distribution is the instantaneous value at a point.

The mutual dependence of cause and effect are exhibited in materials as stress and strain. Force in a body results in deformation. The intensity of deformation is termed strain, and is measured as deformation per unit of length or volume. Stresses and strains are generally referred to as of three kinds —tensile, compressive, and shearing. By tensile and compressive forces are meant those forces which act normally to the surface under consideration. By tensile and compressive stress is defined the intensity of the normal forces per unit of area.

By shearing force is defined the total force which tends to cut a body apart and move its parts relative to each other in a direction tangential to the surface of rupture.

By shearing stress is defined the intensity of force per unit of area which exists across an internal surface when the parts of the body on either side of the surface exert equal and opposite forces tending to cause motion laterally in a direction tangential to the surface.

By frictional stress is defined the intensity of force per unit of area which exists between two external surfaces in contact when the two bodies are subjected to opposite forces tending to cause motion laterally in a direction tangential to the surfaces in contact.

The determination of stresses and strains in materials has formed the subject of many investigations. The usual mode of attacking the problem involved has been often referred to as the "mathematical theory of elasticity," which is, however, an all-embracing title that should rightly cover the whole subject. In its narrower aspect it means the mathematical analysis of stresses and strains in an elastic medium that is homogeneous and non-granular. Seeing, however, that such a material is purely imaginary, the mathematical analysis, though strictly exact in itself, may not conform to the facts of experiment. The mathematical analysis that has arisen from such consideration is a most powerful assistant to the engineer, and by the adoption of suitable functions to suit the peculiarity of the composition of each material, will, within the limits of elasticity, afford many important relations that accord with the facts determined by experiment. Mathematics, in-

cluding, in large part, the processes which have been evolved by consideration of the ideal elastic substance, have only comparatively recently been applied to another mode of approaching the determination of the stresses and strains in materials in which their granular nature is recognized and their resistance and deformations expressed in terms of the inter-molecular forces and motions. Prof. Sir J. A. Ewing has put forward in a tentative way a theory of electrical attraction between molecules, while Prof. A. Rejtö, of Budapest, has worked out in considerable detail a theory dependent upon molecular friction.

Various hypotheses to account for rupture in bodies under stress have been advanced by writers from the point of view of the mathematical relations of a homogeneous, non-granular elastic material. Lamé assumed that material was ruptured when the greatest tensile stress reached a limiting value. Poncelet, followed by Saint-Venant, assumed that failure occurred when the greatest extension reached a certain limit. Tresca, followed by G. H. Darwin, suggested that rupture occurred at a limiting value of the largest difference of the greatest and least principal stresses, while Coulomb made a somewhat analogous suggestion that failure occurred when the greatest shear reached its limiting value. This latter view, in recent years, has received much support from experiments conducted by Mr. J. J. Guest, with the consequence that it is often termed "Guest's Law."

Defining force as any circumstance that changes or tends to change a body's state of rest or of uniform motion in a straight line, and accepting Newton's Second Law of Motion, that change of motion is proportional to the impressed force and takes place in the direction of the force, we note that this law makes no mention of any other forces that may be acting on a body at the same time, so that it follows as a corollary that every force, however small, produces its whole effect, however great may be the other forces acting on the body, a fact which is subject to experimental verification. Therefore we may get the same effect by replacing a number of forces acting upon a structure by other fewer ones (called resultant forces), or, conversely, by finding a larger number. When these

latter are chosen to be in particular directions acting parallel to established ones they are referred to as component forces. Generally these components are related to two or three axes at right angles to each other. Forces may be combined together algebraically, and a common way is to make the algebraic sum of their trigonometrical components. Conversely we may take the resultant forces and analyse them into their components normal and tangential to any plane.

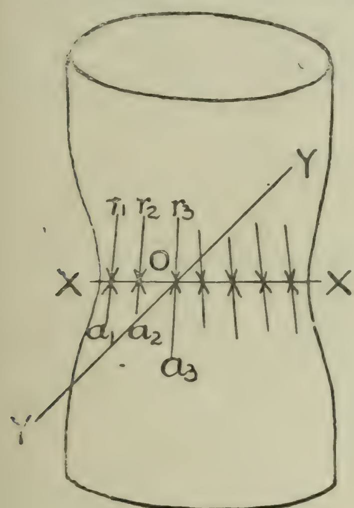


FIG. 1.

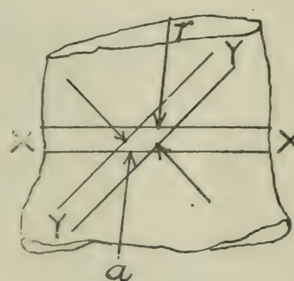


FIG. 2.

Appendix I relates to the resolution of a force into its rectangular components and establishes certain relations between the components.

The stresses in a homogeneous, non-granular, elastic solid may be examined by various mathematical expedients. Graphical aids are, however, of much assistance to simple, clear explanations, and there are two common modes of representing the stress at any point of a body. One method of representation is by means of a parallelopiped enclosing the point, whose faces coincide with the axes under consideration and upon the faces of which stresses are applied, the parallelopiped being so small as to enable the stresses to be considered as sensibly constant in intensity. The

other method of representation is to take intersecting planes coinciding in direction with the axes to be considered, and representing the stresses in the neighbourhood of the point of intersection by arrows impinging upon the planes. This will be made clearer by reference to Fig. 1, which represents only two planes, namely, x x, and y y. The stresses from point to point along the plane x x are shown by the arrows a_1, a_2, a_3 . . . r_1, r_2, r_3 representing actions and reactions which vary in intensity and obliquity as shown. Now, if the other plane y y pass through O (the origin), there will similarly be pairs of actions and reactions at all points of it, and a pair of definite

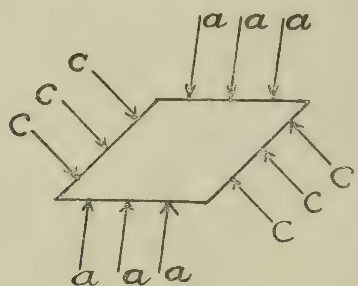


FIG. 3.

intensity and direction at O. Now let us consider x x as an infinitely thin layer of the material with a_1, a_2 acting against the under surface, and r_1, r_2 reacting against the upper surface. Similarly let us treat y y as a thin layer in the same fashion. Representing this to a magnified scale we have Fig. 2.

Singling out in Fig. 3 the small element containing O, we have the parallelopiped upon the faces of which we put arrows to represent the stresses at the point O, these stresses being constant in intensity because the surfaces of the parallelopiped are so indefinitely small that the stresses over them cannot vary sensibly from the intensities of the stresses at the point O. Yet being stresses and not forces we still find it convenient to continue to represent them by sets of equal and opposite arrows a and c. For

simplicity of analysis the kind of parallelopiped generally chosen is a cube.

Alternatively, instead of a parallelopiped, we can show the stresses at the point by arrows on small portions of the intersecting planes as in Fig. 4.

Certain fundamental mathematical relations may now be referred to. Firstly, there is the proposition that if the stress on a given plane in a body be in a given direction, the stress on any other plane parallel to that direction must be in a direction parallel to the first-mentioned plane (for proof, see Appendix II). A pair of stresses each acting as a plane

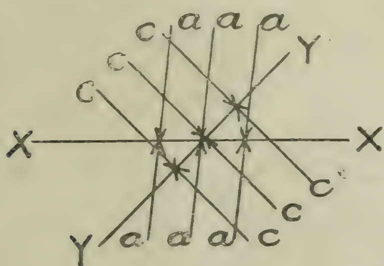


FIG. 4.

parallel to the direction of the other were termed by Rankine "conjugate" stresses. In a rigid body the stresses may be independent of each other, and of the same or opposite kinds. In a fluid substance the stresses cannot thus be independent nor in a granular substance devoid of cohesion.

Appendix III contains proof of the proposition that if there is tangential stress on any plane it is accompanied by tangential stress of equal intensity on planes transverse thereto.

This relation leads to a further analysis, given in Appendix IV, whereby it is found that the stress at any point can be completely specified by merely six components on rectangular axes, three being normal and three tangential to the three planes in which the three axes lie, and, further, that three planes can be determined upon which the resultant stresses are wholly normal. Such planes are called Principal Planes, and

the stresses across such planes are called Principal Stresses. The directions of the principal stresses are generally called the axes of stress.

In many practical cases there is no appreciable stress across one of the principal planes, and the statical analysis becomes two-dimensional with corresponding simplicity. Some of these simple two-dimensional results are given in Appendix V.

Particular attention may be directed to the latter portion of Appendix V, where the formulæ for determining the maximum shear stress are given. These are of importance in view of Guest's theory. In an appendix to Prof. C. A. M. Smith's "Handbook of Testing Materials" (Constable), there is an extended account of various experiments on steel and cast-iron specimens under combined stresses, which clearly seem to show that when rupture occurs the maximum shearing stress is far more constant for ductile materials than the maximum principal stress, but not so for a brittle material like cast iron, where the reverse order of things applies. Brittle materials are weak in tension, and the author of this paper offers the opinion that rupture of ductile materials occurs by shearing, but that rupture of brittle materials occurs by tension. Rupture by compressive stress alone never occurs. If materials are ruptured under a thrusting load it is by the shear or the tension that is induced thereby, as will be referred to in greater detail hereafter. It is suggested that ductile materials are distinctive in their structure in that they are crystalline in structure and appear to break down by sliding along the cleavage planes of the crystals. To quote from Prof. Sir J. A. Ewing's "Strength of Materials" (Cambridge University Press), second edition, p. 46:

"When a metal is polished and lightly etched it is seen under the microscope to consist, in general, of crystalline grains, which are crystals with irregular outlines, the form of the boundaries having been determined by the meeting of the grains in their growth. Within each grain there is a definite orientation of the elementary pieces of which the crystal is built up, and the orientation changes from grain to grain. When the metal is stretched by pull or by cold rolling or cold hammering, the grains become elongated. But when

the piece, after the treatment, is heated to redness, and is again examined microscopically, the grains are found to have reformed and to be on the whole as long in one direction as in another. Slow cooling tends to produce large grains, and fast cooling produces comparatively small grains.

"Microscopic observations by Mr. W. Rosenhain and the author have demonstrated that the manner in which a metal yields when it takes any kind of permanent set is by slips occurring on cleavage or 'gliding' surfaces within each of the crystalline grains. These slips show themselves on a polished surface by developing systems of parallel lines or narrow bands, each of which is a step caused by one portion of the grain slipping over the neighbouring portion. Two, three, and even four systems of slip lines may be traced when the metal is considerably strained. Plasticity results from these slips, although the elementary portions of the crystals retain their primitive form and the crystalline structure of the metal as a whole is preserved. In some metals, in addition to simple slips or motions of pure translation, there is a molecular rotation resulting from strain, which gives rise to the production of 'twin' crystals. Apart from this, however, the occurrence of slips on three or more planes within each grain suffices to allow the grain to change its form to any extent as the process of straining proceeds."

It is a fact commonly noted that chilling steel will make it brittle. The smaller crystals then formed would appear to be of the nature of grains separated by films or membranes of material of different composition. This is so with cast iron, and fracture appears to occur in such cases by rupturing the surrounding films, which are composed of material of cementitious nature.

A lengthy account of Prof. Rejtö's analysis of stresses and strains is given in an appendix, because his theories, though of considerable importance (especially as regards ductile materials), have not so far been generally available in this country. He has carried his studies much farther than this preliminary account would show, especially in connection with shearing and punching of plates, etc. Further reference by

those especially interested should be made to a paper in French from which the account in the appendix is taken, and read some years ago before the Stockholm Congress of the International Association for Testing Materials; also an article translated into English in the journal *Les Matériaux de Construction,* Nos. 19-22, 1904, the official journal of the International Association, and an English translation of an article on bending stresses and strains in the *Proceedings of the International Association for Testing Materials,* vol. ii, No. 10, June 28, 1912. The characteristic shear

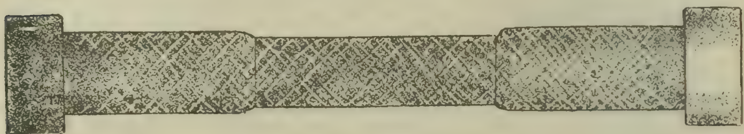


FIG. 5.

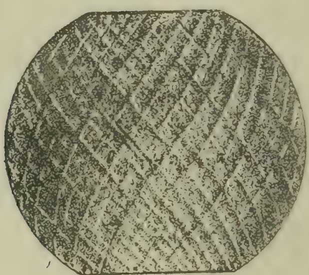


FIG. 6.

slipping of sections in ductile materials, explained at length by Prof. Rejtö, is shown in Figs. 5 and 6, which illustrate tension and compression specimens that have been strained. The ridges on the surface resulting from the slipping greatly support the theory for such action.

The writer, working on his assumption that brittle materials of distinctly granular structure fail by weakness in tension, has made an analysis of the conditions of rupture of a granular mass in which there is cohesion. The analysis proceeds on allied lines to Rankine's analysis of frictional stability, with the differ-

ence that cohesion is not ignored. Appendix VII gives the argument, and attention is directed to the formulæ for the resistance of materials subjected to crushing. It is a well-known fact that with the thrusting of building stones, mortars, and concrete, as also of cast iron, rupture takes place by sliding on oblique surfaces.

Various analyses of the stresses existing under such a mode of failure have been put forward by writers on the strength of materials. Arguing from the relation which is given in formulæ of Appendix V, it has been supposed that brittle materials under stress should fail on planes inclined at an angle of 45 degrees to the direction of the stress. With ductile materials under direct tension the analysis appears to be approxi-

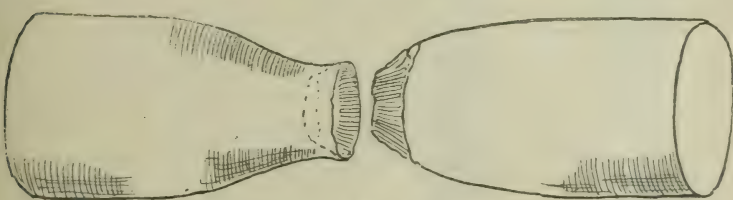


Fig. 7.

mately correct, though Prof. Rejtö contends that the angle of slipping is a characteristic property of the materials and varies with different materials. It is well known that tension specimens draw out at the point of rupture into a conical and cup-like form of fracture, as shown in Fig. 7, the angles at the faces of the cup being approximately 45 degrees. In compression, however, with ductile materials the relation does not appear to hold with the same degree of accuracy, the very nature of ductile materials being to swell in compression, and as they swell so their area increases. The stress cannot be increased beyond a certain point, at which they become permanently deformed, according to geometric laws, so that there is no indication of elasticity or tendency to return to the primitive shape. This M. Tresca called the pressure of fluidity. For mild steel its value is about 50 tons per square inch, and the form that the compression-block takes is shown

in Fig. 8. By microscopic inspection of the faces of such compression-blocks a justification of the ordinary analysis is, however, found in that ridges are noticeable upon the surfaces showing where the material has slipped upon the cleavage planes of the crystal structure, which slippings occur approximately at 45 degrees. As shown in Figs. 5 and 6, Prof. Rejtö's analysis seems to be a reasonable one as regards ductile materials, but judgment ought to be reserved as regards its application to brittle materials until more experimental evidence is forthcoming.

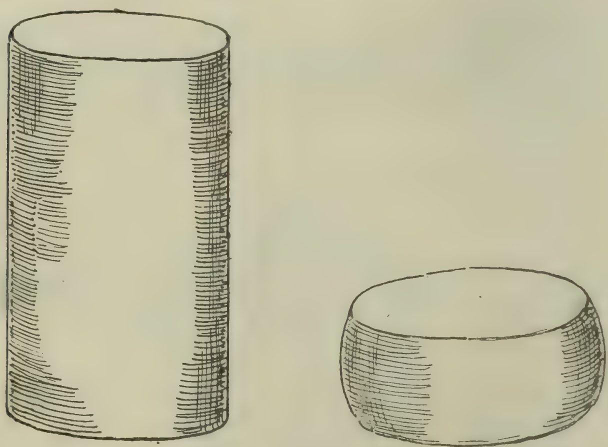


FIG. 8.

The customary mode of analysis proceeding on the lines of the mathematical theory of elasticity for non-granular and homogeneous elastic solids, as has been said, leads to a reasonable explanation as regards ductile materials. Navier made a further assumption in an attempt to explain why brittle material did not fracture on planes at angles of 45 degrees, but, as with cast iron and concrete, at some angles inclined in the neighbourhood of 30 degrees to the axis of the compressive stress. His assumption was to consider that materials failed by sliding on an oblique plane, and that the friction on that plane had to be overcome as well as the shearing resistance of the material. One

of the best demonstrations of this mode of analysis is to be found in Prof. J. B. Johnson's "Materials of Construction" (Chapman and Hall, Ltd.), in which experimental data are given intended to support Navier's mode of analysis. The author of this paper was led to his other mode of analysis whereby the material was considered as a granular mass with cohesion, by knowing that later experiments did not appear to agree with the values derived by Prof. Johnson's analysis. Experiments had, indeed, shown that the resistance of concrete to direct shear was very different from the shearing value that was found by Navier's analysis, and punching experiments on concrete specimens again showed different values. The shearing value for concrete determined by different modes of analysis and experiment were found to be different. It remained, therefore, to see if some reasonably uniform relation could be found between the resistance to shearing force derivable from experiments on direct shear, and from the analysis of the shear stress induced by direct compression. This relation appears to consist in the dependence of the resistance to shear upon friction and cohesion, or, as the latter might equally well be termed, tension. A uniform relation is found between the experiments on building stones, concrete, and cast iron in respect to failure under tension, compression, and shear as is shown by the references in Appendix VII.

That failure of materials cannot occur by real compression is fairly evident when we consider the fact that even a liquid such as water is practically incompressible. If we apply force from every direction, we do not cause rupture, as the material is not able to escape laterally. A number of experiments were made some years ago at McGill University by Prof. Frank D. Adams, and described in the *Proceedings of the Royal Society,* in which marble was placed in steel cylinders and compressed. It was found that this brittle material could be subjected to very great deformations without much reduction of its strength against thrusting stress, the material, indeed, showing a ductile character and ability to undergo plastic deformation when adequately supported in a lateral direction. The same thing has been found by experiments

on concrete, as, for instance, some classic experiments by Prof. Ira H. Woolson. In one concrete in a steel cylinder was bulged out under compression into the shape shown in Fig. 9, yet when the encasing cylinder of steel was removed, the strength was found not to be appreciably different to that of concrete which had not been subjected to such deformation. It would seem that when complete lateral restraint is imposed, such as by enclosure in a cylinder, the cohesion is not developed, but, instead, the crystalline or directional substances undergo distortion by slipping on cleavage planes as a strictly ductile material would ordinarily.

The late Monsieur Armand Considère, in one of his papers on the resistance of hooped concrete, referred to experiments in which concrete had been placed

FIG. 9.

to pass through the box of a hydraulic ram so as to be subjected to lateral pressure, and at the same time was subjected to vertical pressure. It was found that great crushing loads were thus enabled to be sustained without rupture. He also referred to an experiment in which that brittle material—glass—could be bent permanently without fracture when placed in a liquid under great hydraulic pressure. From this he drew the conclusion that the efficiency of hooping in increasing the crushing resistance of concrete was in the nature of a lateral support such as might be given by uniform lateral hydraulic pressure. The writer, however, thinks it would be more legitimate to regard the action of hooping in a different way. When compression specimens are made more dumpy their resistance to thrusting stress is increased, and the reason is assumed to be that failure with such specimens is not permitted to occur on the ordinary

plane of rupture, which is inclined between 20 and 30 degrees from the vertical, or 60 and 70 degrees from the horizontal, because of the lateral restraint imposed by the friction on the bearing plates of the testing machine. We know that when no lateral restraint is given on these bearing surfaces the material ruptures much sooner. For instance, in Prof. W. C. Unwin's "Testing of Materials of Construction" (Longmans & Co.) illustrations are given showing different modes of rupture of stone bedded upon various materials. In the case of lead seating, the squeezing of the lead is shown to have ruptured the block vertically by tearing it asunder. Lead, for such reasons, is not a good material to use for bedding, and some experiments recorded in a paper by Prof. H. I. Hannover, of Copenhagen, published in the *Proceedings of the International Association for Testing Materials*, Vol. II, No. 11, July 1912, show that the pressure on a lead block varies, due to lateral escape of the lead in conjunction with friction on the bed plates. No doubt local deformations and inequalities both of the block and of the bed, together with non-axial thrust, result in a variation in the pressure across the bearing area in test-pieces of any material, and thus it is advocated that in order to obtain the true crushing resistance of brittle material, the length of test specimens should be at least three times their breadth. In addition, it might be advisable to let the bed-plates of the testing machine rest upon double knife-edges both top and bottom, which have means of moving, by a small amount, the test specimen on its bearings, after putting on a trial load for adjustment purposes, in order to ensure that the axis of thrust may be made to coincide with the real axis of the piece so as to eliminate bending, somewhat in the manner suggested by M. Considère as referred to on page 361 of Johnson's "Materials of Construction," fourth edition, for inequalities in the material almost always cause the real axis to occupy a different position from the geometrical axis. As to whether the loading is axial could be determined by observing the deformation on different sides under the trial load by means of a compressometer giving separate readings for the several sides.

Reverting to the function of hooping, the writer would prefer to consider the action to be as follows: The hooping either (1) applies a lateral compression equivalent to hydraulic pressure proportional to its extension under the lateral swelling defined by Poisson's ratio; (2) after initial cracking on the plane of rupture resists lateral motion of the parts by sliding on the plane of rupture until failure occurs by the hooping breaking; or else (3) is of sufficient strength not to be able to be broken in this way, in which case the rupture of the concrete is confined to that extending between two adjoining hoops, that is to say, the concrete fails between the hoops like an extremely dumpy test specimen bedded on concrete. It is possible to calculate the effectiveness of hooping in these three respects.

It is desired to point out that the author has had insufficient opportunity to carry this mode of analysis of cohesive granular substance very far. As to how it would apply to ductile materials, and as to whether it would work out to give much the same result as Prof. Rejtö's theory has not yet been investigated, nor its relation to the resistance of pieces subjected to bending. It is hoped to present further contributions to the subject at a subsequent date.

It is now proposed to deal with certain aspects of shearing forces and shearing stress in connection with pieces subjected to bending.

Appendix VIII refers to the relations between the bending moments and shearing forces on beams, and also the effect on the distribution of shear stress in beams of the customary theory, commonly referred to as being based upon the conservation of plane sections (the supposition that plane sections remain plane after bending) though it is unnecessary to make such assumption in order to derive the ordinary beam equations, such as radius of curvature, shear distribution across the section, etc. In advanced works on the mathematical theory of the elasticity of materials it is demonstrated that the fundamental relations between stress and strain do not warrant the assumption that plane sections remain plane after bending, nor do they warrant us adopting straight line functions in deriving the ordinary beam equations above referred to. If we consult Prof.

A. H. Love's treatise, entitled "The Mathematical Theory of Elasticity" (Cambridge University Press), we find that the full theory is certainly intricate, and likely to present considerable difficulties to many engineers who are not gifted in the direction of liking mathematics, and have not had opportunities of acquiring that knowledge such as are so adequately afforded students of engineering to-day. A full demonstration is really too intricate to be possible of adoption in practice, even if we simplified its application by the compilation of diagrams. Further, the mathematical theory is based upon the assumption of the ideal character of the material, and applies only within limits of elasticity, so that for both reasons it does not strictly apply to materials in practice which are not uniform in structure and are not truly elastic, undergoing even at small stresses a certain amount of plastic deformation. The mathematics, however, are strictly exact if we grant the assumptions made, and writers on the subject have pointed out that equations are thus obtained which tend to show that the natural distribution of the shear across the beam is not of the kind derived by the ordinary mode of investigation, where warping of the sections by horizontal shear in beams is not taken into account. The latter, however, gives results so sufficiently close to the exact ones under average conditions, that it is well to adopt it for simplicity. Even if we take the distribution of shear stress according to the ordinary methods we still find that even with those assumptions we are compelled to recognize that the fibres of a beam undergo relative distortion by shear, the angular movements of the layers being greater as the shear stress increases, and being greater between the extreme fibres and the neutral axis than at either of those places. In a paper by Mr. Henry S. Prichard, published in the *Transactions of the American Society of Civil Engineers*, Vol. LXXV, p. 895, the effect of the distortion of the sections of a beam due to shear is very well demonstrated. Fig. 10 is taken from his paper and can be explained by reference to Appendix III, whereby we saw that if shear occurs on one plane it is accompanied by equal shear on other planes inclined thereto, while at the end of

Appendix V it is shown that shear forces cause angular distortion converting rectangles into oblique parallelograms. The increase on one diagonal of a rectangle and the shortening on the other is cumulative from the neutral axis of the beam outwards, so that the sections are gradually moved apart from each other until towards the ends the original plane section has taken the shape of an old English letter ʃ. In analysing the effect of this Mr. Prichard made a simplifying assumption by

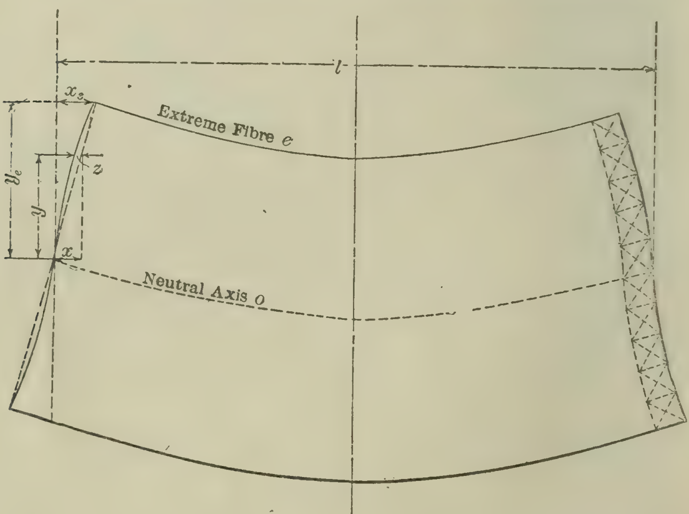


FIG. 10.

taking the distribution of the shear stress across the section to vary, as derived in the customary simple way referred to in Appendix V. In the discussion of the paper, Mr. J. P. J. Williams gave an analysis which consisted of an extension of Mr. Prichard's, and deserves to be studied. In Mr. Prichard's paper, and the discussion thereto by Mr. Williams, it was demonstrated that the ordinary formulæ are found by this improved analysis to be in error varying in amount for various forms of section. With very short rectangular beams of comparatively great depth the error in the calculated stress amounts to something very

considerable, and for various types of plate-girders and joists the error, though less, is serious. As the ratio of span to depth increases the error gets less. Finally, one comes to the conclusion that, except for very short beams, the customary mode of calculation is not seriously in error.

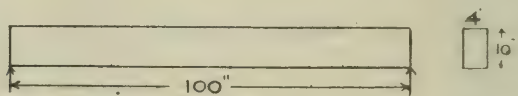
There is another point which needs to be considered in connection with the shear in beams of ductile material. Reference has already been made to the fact that experiments tend to show that rupture of ductile materials under compound stress occurs by shearing stress on oblique planes. We ought, therefore, in designing beams, to have regard to the point where maximum shear will occur. This is often calculated for vertical or horizontal planes, but it requires determination on oblique planes. Prof. Robert H. Smith, in *The Builder's Journal and Architectural Engineer* for August 15, 1906, worked out the effect of this on two joist sections, from which it appeared that if the web were fairly slender the maximum intensity of shear stress was found at the connection between the web and the flange and not upon the extreme fibre. This points to the conclusion that our standard sections ought to be investigated with regard to their possible weakness at the shoulders, as such weakness might be very serious in beams of short span.

It is well known that webs need to be stiffened by the provision of vertical stiffeners under point loads to prevent crushing and buckling of the web, and the provision of stiffeners to prevent the rib buckling by diagonal stress is also adopted more or less empirically (seldom by theoretical determination) in the case of plate girders.

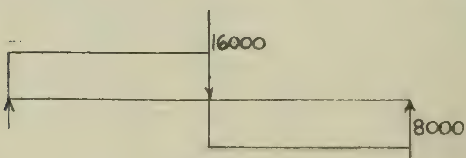
Prof. W. E. Lilly some years ago carried out experiments in this respect upon plate girders; and he drew the conclusion from the fact that the directions of the principal stresses crossed each other at right angles, at an inclination to the neutral axis of about 45 degrees, that the web was in the condition of a strut, so that stiffeners might require to be provided either in the form of vertical or inclined members in order to convert the plate girder into a lattice girder. Prof. Lilly's studies are recorded in his book on "The Design of Plate Girders and Columns" (Chapman

and Hall, Ltd). Appendix VIII relates to calculations for such parts.

In reinforced concrete we have a somewhat analogous position. Reinforced concrete beams are quite a distinct type in themselves, because they pass through two stages in their resistance to cross-breaking. In the first stage the steel remains uniformly connected to the concrete in which it is embedded. In the second stage the steel slips through the concrete for part of its length and the concrete cracks in the tension portion, so that the beam becomes converted into a form of construction which, according to different modes of reinforcing, would appear to consist of either a con-



Elevation. Cross-section

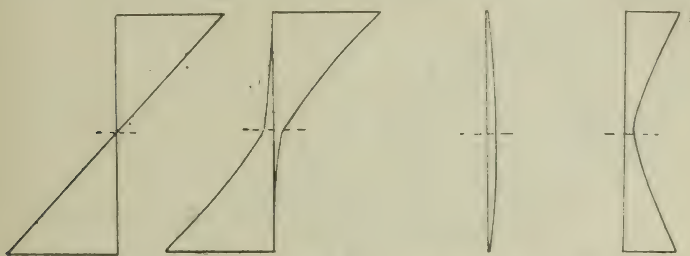


Shear Diagram.

FIG. 11.

cealed arch with a steel tie, a trussed beam analogous to the trussed timber beam, or a frame. Frames subjected to transverse loading are, of course, subjected to shearing forces, but the effect of the shearing forces is naturally very different on frames from that on a girder with a solid web or upon arches. A shearing force diagram only consists of the plotted values of the tendency to shear it asunder which the load exerts upon the structural member. We can, therefore, have a shearing diagram equally applicable to a beam, an arch, or a frame, but the manner of using the diagram for determining the internal stresses which result from the shear forces must be different in each case. Supposing, for instance, we take the shearing diagram shown

in Fig. 11, in relation, firstly, to a beam as there illustrated. We know from Appendix VIII that the resultant shearing stresses are distributed across the vertical section, somewhat in parabolic manner, being of greatest intensity at the neutral axis, while the flexural stresses of compression and tension vary across the section in a triangular manner, being of greatest intensity at the extreme fibres. If we calculate the resultant maximum shear stress on oblique planes, and the principal stresses on oblique planes, we find a different distribution in the nature of that shown in the diagram,



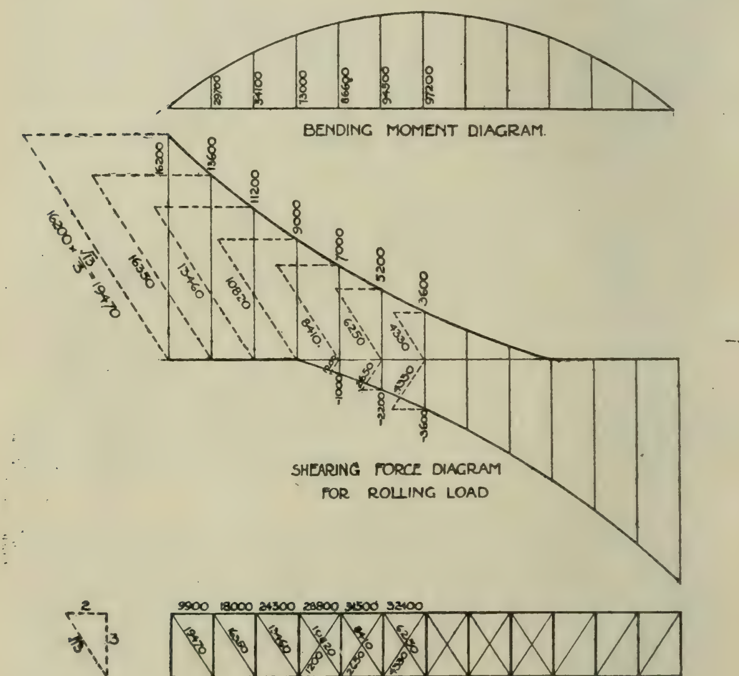
Stresses normal to vertical section. Principal stresses on oblique sections. Stresses tangential to vertical section. Maximum tangential stresses on oblique sections.

FIG. 12.

Fig. 12. If the section of beam were not rectangular, but had a web the intensity of shear stress on oblique planes would be more uniform in intensity throughout the web and may even be greater in intensity than on the outside edges. Now let us consider the application of the same shearing diagram to a rectangular frame. The stresses in the members may be conveniently calculated by means of the "Method of Sections," which becomes the same as taking the vertical shearing force at every section and determining its component in the direction of the diagonal members of the truss as shown in Fig. 13.

Then, again, the action of such shearing force on an arch is to give a line of thrust in the arch ring, which, where this departs from the centre line of the arch ring, will cause eccentric stresses that result in the direct stress varying across every section of the arch ring

that is taken perpendicular to its centre line, and that result also in a shearing force equal to the component of the inclined thrust tangential to the radius of curvature. Here, again, the maximum shearing stresses on oblique planes and the principal stresses are very different from those of the truss and beam. At the same



TRUSS

FIG. 13.

time, one may be misled in connection with shearing force diagrams as applied to frames by reason of the similarity between the use of the diagram for calculating stresses in a beam and frame. Whereas the application is close in the case of concentrated loads applied at the connection points of the frame, in the case of distributed loads the shearing force diagram

is not so closely related. For instance, an evenly distributed load will give a shear diagram as shown in Fig. 14. Now a beam with a solid web would have its stresses varying continuously throughout the length just as the shearing force varies throughout, though the order and rate of variation may be different ; but in the case of a truss the distributed load may be considered to cause secondary bending in the compression or tension members (whichever directly receives the loading), and these members may be considered to concentrate the load at the connection (panel) points of the truss. Therefore, as regards

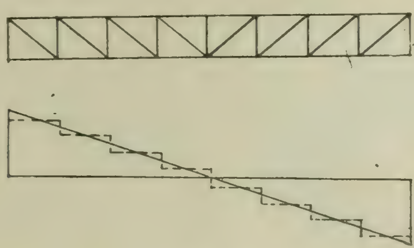


FIG. 14

the determination of the stresses the shear force diagram that we ought to use will be of the form shown in the dotted curve of Fig. 14, which is operated upon by finding components parallel to members of the truss the same as before referred to. The effect of the distributed load between the panel points is to cause bending in each member, as though it were sustained as a beam or bar at the connection points, and the bending moments induced in this way will, of course, depend upon the degree of continuity or fixity in such members, thus causing the compression and tension members to undergo variation of stress across their section. A shearing stress in the booms will also result from such distributed loading ; the compounding of this shearing stress with the direct stress can be employed to determine, as before, the maximum stress on oblique planes, both for shear and for the principal stresses of tension and compression.

The general relation between the shearing force

BEAMS WITH TRIANGULAR LOAD.

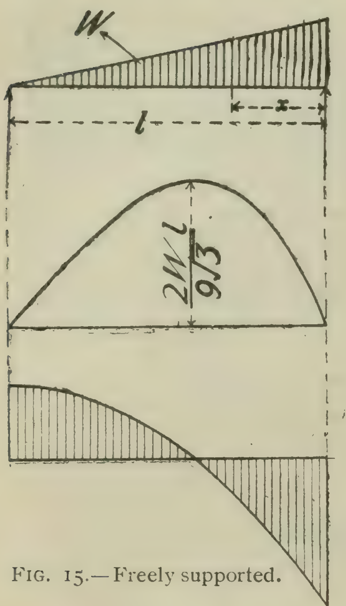


FIG. 15.—Freely supported.

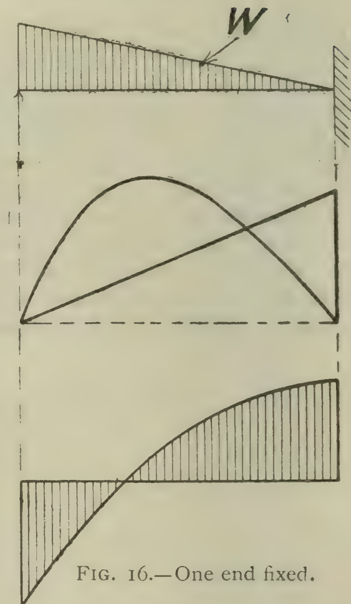


FIG. 16.—One end fixed.

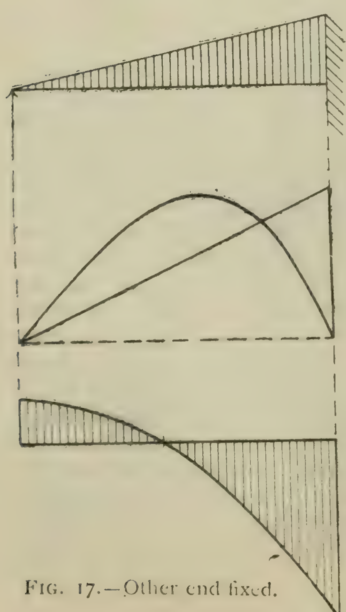


FIG. 17.—Other end fixed.

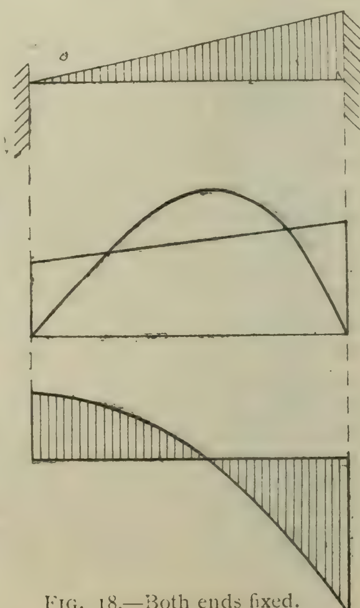


FIG. 18.—Both ends fixed.

diagram and the bending moment diagram for homogeneous beams is dealt with in Appendix VIII. The fact that the area of the shear force diagram up to any point is equal to the numerical value of the bending moment at that point in the case of freely supported beams will often be found of considerable service in practice, while the feature that the point of maximum bending moment is the point of zero shear, which is of general application to free, fixed, and continuous beams, is of common employment for the determination of the maximum bending moment.

Figs. 15–18 show the effects of fixing both ends and one end or the other upon the distribution of the shear along the length of a beam sustaining a triangularly distributed load increasing in intensity from one end to another, such as would be given by earth or water pressure. The ordinates to the bending moment and shear diagrams for a freely supported span are given by the formulæ for the point x—

$$B_x = \frac{W}{3}(l - x)\left[1 - \frac{(l - x)^2}{l^2}\right]$$

$$S_x = W\left[\frac{1}{3} - \frac{(l - x)^2}{l^2}\right] \text{ or } \frac{W}{3l^2}(6lx - 3x^2 - 2l^2)$$

The fixing moments and reactions at the ends for other conditions shown in Figs. 16, 17, and 18 are as follows :—

Fig. 16—

$$B_c = -\frac{7Wl}{60}$$

$$S_L = R_L = \frac{11W}{20}$$

$$S_R = R_R = \frac{9W}{20}$$

Fig. 17—

$$B_c = -\frac{2Wl}{15}$$

$$S_L = R_L = \frac{W}{5}$$

$$S_R = R_R = \frac{4W}{5}.$$

Fig. 18—

$$B_{cL} = -\frac{W\,l}{15}$$

$$B_{cR} = -\frac{W\,l}{10}$$

$$S_L = R_L = \frac{3\,W}{10}$$

$$S_R = R_R = \frac{7\,W}{10}$$

The open bending moment diagrams are formed by drawing straight lines from the end fixing moments above set forth upon the diagram for the freely supported condition, the portions above and below the straight line being the residual or final bending moment diagram.

The maximum bending moment for Fig. 15 is marked on the diagram and occurs at the point of zero shear, which occurs at a distance $l-x=l/\sqrt{3}$.

The shaded shear curves of the figures are merely formed by shifting the horizontal line up or down to give the amounts of the reactions above set forth. Partial fixity as occurs with continuous beams will affect the shear in similar fashion, and it should be noted also that partial loading of continuous beams (i.e. loading any or all of the spans wholly or in part) will result in different distributions for the shear and gives different reactions on the supports. For this reason the current practice of generally taking the load on the support next to the end of three or more continuous equal spans as equivalent to the load on one span is considerably in error, as is the practice of taking the shear as the same as on a freely supported span.

For the determination of shear throughout the length of beams required to sustain travelling loads where these point loads are applied at several points of the span and can move throughout the span, use is made of influence line methods as explained in Appendix VIII. The forms of the bending moment and shear diagrams resulting from the application of a point load on a freely supported span are shown in Fig. 13,

which also illustrates how the forces in the diagonals may be obtained from the shear diagram, while the forces in the booms are obtained from the bending moment diagram. By the process given in Appendix VIII the position of the load which will give the maximum shear at any point can be readily determined, and the shear curve is the plotting of the maximum shearing forces throughout the beam that result from the system of rolling loads.

In Appendix VIII reference is made to the manner in which the shear in the webs of beams is applied to the flanges. This may be considered generally as a gradually increasing thrust from the ends of a beam and tends to cause lateral flexure of the compression flange considered as a long strut.

When the web is stiffened either vertically or obliquely, the stiffeners act as struts, and the web takes diagonal tension like the diagonals of a frame. This diagonal tension tends to cause vertical secondary bending of the flanges between the panel points, which should again be taken into account in the detailing of plate girders.

If the stiffeners are arranged vertically, they convert the girder practically into a Pratt truss, where the stiffeners are in compression. If the spacing of the stiffeners is closer together than the arm of the beam apart, the diagonal tension may still be considered as inclined at 45 degrees, the system becoming one of superimposed trusses. If the stiffeners are diagonal the girder becomes an N truss. In riveted joints not only the shearing resistance of the rivets, but the safe-bearing stress on the plates or other members through which they pass must be studied. Very often the bearing stress is less than the shearing resistance, the value of which is derived in Appendix V. In using that value, however, it should be remembered that the holes in which the rivets are inserted are punched or drilled large enough to enable the rivets to be inserted easily. Consequently, when the rivet is closed it will have the larger diameter of the hole.

The design of riveted joints, if there be no bending, is so simple that the matter need not be touched on further here, except to remark that experimental investigations have shown that in group riveting some rivets

always come into play before others, and are thus strained beyond their yield point, but that never affects the ultimate strength, because the yielding puts the stress more evenly on the other rivets, so that they all become effective in the end. The reliance in practice on such joints would seem to be overlooked by engineers who have accepted as gospel the modern idea that the factor of safety should be determined, not upon the ultimate resistance but upon the yield point.

Reinforced concrete beams are quite a special form of construction, because the stresses are induced in them in various ways according to different modes of arranging the reinforcement, and according to the amount of reinforcement; that is to say, a reinforced concrete beam may prefer to act as a flat arch with a tie rod if it is able to hold up in such form with less stress than it would in some other fashion. On the other hand, if it acts as a beam, the conditions are different in the case where the stress is such as not to have ruptured the concrete by tension therein from the case where the concrete is cracked. Generally, some parts of all reinforced concrete beams are in the uncracked condition, and really require analysis on the lines of a homogeneous section, the materials, however, having different moduli of elasticity both for the concrete in tension and compression and the steel. In the majority of cases the reinforcement is insufficient in amount to prevent cracks occurring at some point or points in beams, and then when cracks appear it means that the steel has moved in the concrete, and that the construction either consists of a trussed beam or a framed beam in part or in whole. Where the reinforcement is inclined at the ends as shown in Fig. 19, there is an obvious analogy to a trussed timber beam. Such a type of reinforcement is quite appropriate for point loads. When the cranked bars are in combination with straight, horizontal bars, a practical type of reinforcement is provided which forms a sort of halfway-house, and can equally well resist point loads or distributed loads. In the latter case not only do the straight bars produce one sort of beam action, but the cranked rods resist the shear in that the inclined pull in the bars affords a vertical component

to resist the shear. If P is the pull in the bar, obviously the vertical shear resisted by the vertical component will be $P \sin \theta$, while the shear to be taken by the bar is $S \operatorname{cosec} \theta$, where θ is the angle of inclination to the horizontal. If the steel or the top of the concrete were curved parabolically, the vertical component throughout would vary exactly

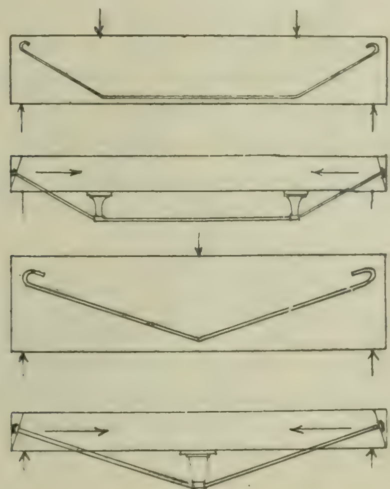


FIG. 19.

as the shear under evenly distributed loads, and no web steel need be provided. Such a type of shear reinforcement implies that the shear is taken only by the steel because a crack may be developed by the concrete. Such cracks may, of course, be so minute as not to be visible, otherwise, if the concrete were not ruptured, by the method in Appendix IV we could find diagonal tension in the concrete, and the stress in the steel would then simply be the comparatively small amount proportional to the ratio of the differing elasticities of the two materials, steel and concrete, E_s/E_c. Of course if the concrete in tension is so

highly stressed as to arrive at nearly its plastic stage, or, at any rate, near its ultimate strength, the elastic modulus of the concrete will be so small as to render the modular ratio E_s/E_c very high. This, however, amounts to much the same thing as a crack, and any vibration would induce complete separation.

Cracks that develop in reinforced concrete beams owing to the excessive stretching of the concrete generally follow the lines of the principal stresses, and point to the conclusion that they are the outcome of the diagonal tensile stresses that become developed in a homogeneous structure in the manner referred to in Appendix V. The very presence of such cracks renders illogical the analysis of the distribution of the shear stress in reinforced concrete beams, which, I believe, was originally put forward in Germany, and appears in Prof. Emil Mörsch's book on "Der Eisenbetonbau," and is also summarized in an appendix contributed by Mr. William Dunn to the British Joint Committee's Second Report on Reinforced Concrete; this analysis depends upon the section being homogeneous, and is based on the principle that the shear on vertical planes is accompanied by an equivalent shear on horizontal planes. The shear is found by that analysis to be of uniform intensity below the neutral axis of the beam, because no normal stress is assumed to act upon the concrete below the neutral axis, and omitting this factor in the equations for shear in homogeneous beams gives the result stated. Seeing, however, that if there should be cracks in the beam extending practically to the neutral axis, obviously there can be no such horizontal shear stress across the cracks, and if there is no horizontal shear stress therefore vertical shear stress cannot exist there either. Indeed, the resistance to the shearing force of a beam must, in the presence of cracks, be altered to the manner in which an arch, a truss, or a frame resists shearing force. Diagonal cracks in nowise preclude the concrete from taking diagonal compression parallel to such cracks.

It was early found in practical work that it was advantageous to reinforce beams against shearing force in other manners than by trussing members. Either

alone or in combination with such inclined reinforcements, web members (either vertical or diagonal) were provided. Numerous patents have been taken out for various forms of separate web members, though often they can be economically provided by turning up the main bars. Where other practical and theoretical conditions preclude one from using so many bars as would enable one to turn the bars up at the proper places, so as to do without the necessity for providing additional separate web members the system may be a double one.

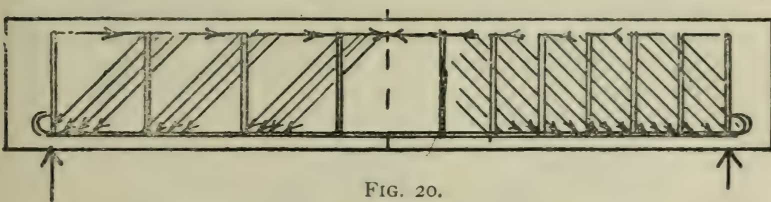


FIG. 20.

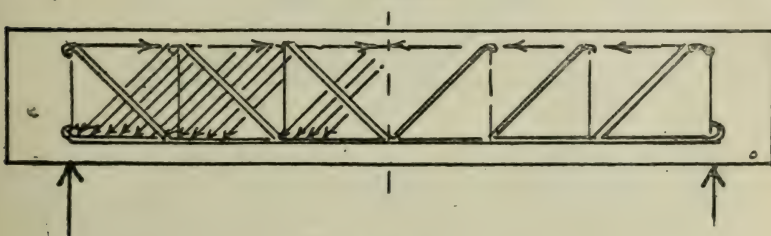


FIG. 21.

The frame action of vertical and inclined web members generally belongs to two types, such as is diagrammatically shown in Fig. 20. In the first case where vertical web members are employed the construction becomes a kind of N-truss, in which the concrete forms compression members and the steel the tension members. In the diagonal form the construction becomes practically a lattice girder. In both types of construction the frames may be, and often are, superimposed; that is to say, the spacing of the vertical members or of the diagonal web members is frequently fairly close, as shown at the right hand of Fig. 20. It is possible by attention to the design

to put the web members very far apart and yet at the same time ensure frame action. In that case the concrete diagonals of the frame would become inclined at such a flat angle that the connection of the concrete to the tension member would not be very efficient unless special means were taken, while the steel would not be so economically arranged. If the diagonal compression in the concrete is at a sharp inclination exceeding the angle of friction, there is a tendency for the concrete to slide along the bar. This may be wholly or partly resisted by grip or adhesion of the concrete to the horizontal members. If, however, the inclination is very flat, this sliding tendency becomes great. In order to provide resistance to slipping, and for economy, the inclination and the stress of diagonals should be kept up, and, furthermore, precautions should be taken to prevent the web members sliding along the main tension member.

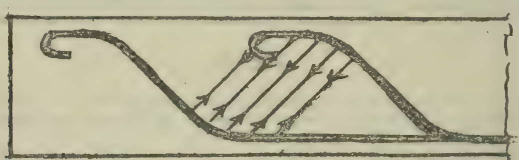


FIG. 22.

If, as has been said, the concrete diagonals are at a flat inclination, the friction induced by the inclined compression will be small, and the compression in the web concrete will come upon a small bearing area. For these reasons the author does not favour, in practice, the use of such very flat trussing, and advocates that web members be not spaced farther apart than the arm of the beam, so as to keep the diagonal compression inclined at 45 degrees, which is the most efficient angle for economizing steel. The whole of the web concrete then becomes effective to take compression, and there is little or no tendency of slipping along the bars. Turned-up bars should have only slow bends, however, in order to give as much bearing as possible, such as shown in Fig. 22.

The author suggests that it is better not to adopt variable spacing of the web members, but to keep

the same spacing throughout, for if we have variable spacing the assumption of a system of superimposed frames, each equally sharing in carrying the load, will not be true owing to the diagonal web members thereof not being all at equal angles, so that we should not be able to say how much was taken by each.

Another disadvantage in spacing web members far apart is that any point load applied between the panel points might cause crippling of the compression boom.

The web members induce compression in the boom by means of their resolved horizontal components, which means that there is a shearing force at every horizontal section of the compression boom, but the presence of the vertical compressive component means that the condition is better than simple, pure punching

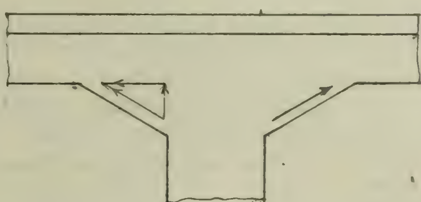


FIG. 23.

shear. The maximum intensity of this may be calculated from the external shear force, because the stress in the diagonal (if at 45 degrees) will be $\frac{S}{2\,a\,b_r}$ where S is the shearing force at any section, a is the arm, and b_r is the breadth of the rib or plane of connection.

Seeing, however, that we shall have pure punching shear at the end of a beam, it is evident that if we designed the beam to be safe against that action we should automatically be secured against shear along the plane of connection between the rib and the slab, so that we may determine the section of a beam by using the formula $b_r = \frac{S}{ds}$ where d is the depth and s the safe resistance to punching shear (from Appen-

dix VII this should be in the neighbourhood of 0·3 c.). In the case of tee-beams the pure punching shear of course would cause the flanges to bend upwards and so the flanges should not be taken into account, but only the area of depth by breadth of rib. It should be noted that there is shear also in a tee-beam along the vertical and longitudinal planes where the slab adjoins the rib, but seeing that there are two surfaces to resist instead of the one horizontal and longitudinal plane, it would be very seldom that any further precautions would require to be taken in practice. The shear along these planes causes variation of the stress across the width and tends to produce tension across the width of the slab over the beam, but the reinforcements and monolithic character of the work offset this.

It should be noted that haunches increase the resistance to shear at the points where it is greatest, not only by increasing the depth and so increasing the section to resist punching shear, but because the inclined compression therein resists the shear in so much as it has an upward component as shown in Fig. 23.

The calculation of web steel becomes very simple when treated as for frames. Thus if the web members are vertical, by analogy with the N girder, we find the average shear on a panel equal in width to the arm of the beam and divide by the permissible tensile stress, so obtaining the requisite area of steel, which can be inserted as one or more stirrups in the length, preferably evenly spaced, as is easily arranged by making them 2, 4, 6, or 8 armed and of $\frac{1}{4}^{\phi}$, $\frac{5}{16}^{\phi}$, or $\frac{3}{8}^{\phi}$ bar. Flat stirrups are not advisable, as they prevent the concrete coming into close contact with the steel at their junctions with the main bars, and so afford risk of corrosion.

In the case of inclined web members at 45 degrees, remembering that there is a concrete diagonal as well as a steel diagonal at each section, we take the average shearing force on a length equal to the arm and multiply it by $\frac{\sqrt{2}}{2} = 0·707$, so obtaining the pull in the steel which is treated as above stated as regards finding area

required. It should be noted that the pull is direct, and consequently the correct stress to employ is that for the tensile resistance and not the shearing resistance as is frequently done.

If bars are inclined at a flatter angle, the author's practice is to take the resistance which they can offer in direct tension and resolve as $T \sin \theta$ and reduce the shear by so much.

If bars are at a steeper angle than 45 degrees, of course we resolve half the shear, obtaining $\frac{S \operatorname{cosec} \theta}{2}$ as the tension. All these three methods of vertical, diagonal, and trussing members can be, and frequently are, employed in conjunction in the same beam.

Punching shear becomes of great importance in the design of footings to pillars, as also does the design of web members. The inclined compression in splayed footings affects the stress computation, but there is no space here now to treat of this which is somewhat of a side issue. The distribution of stresses in splayed sections has been the subject of some speculation, especially in the case of dams.

Sometimes there is considerable torsion to be resisted by reinforced concrete. This torsion affords pure shear if there are no bending stresses induced at the same time. By the methods in Appendix V the resistance to combined stresses may be computed ; and for the reason above explained, the steel being of little help against pure shear stress, the sections should be determined for the plain concrete according to the formulæ for torsion in Appendix V, duly noting that if there are portions of the section so stressed as to be liable to tension cracks these portions need to be omitted from the calculation.

It has been found from tests by Professor Mörsch, which have been confirmed by the author, that the punching shearing stress in reinforced concrete cannot be considered to be resisted to any appreciable extent by members crossing the plane. This is understandable, if we consider the effect of shear on two planks in a compound timber beam nailed together, as referred to in Appendix VIII. The shear would produce bending stress on the nails, and the nails would give a bearing stress on the timber, while before the steel

members crossing a plane in the concrete could act in the same way there would have to be a division by severance, first of all, of the concrete along that plane, otherwise the steel would act merely like a hard stone in the concrete. The section of plain concrete must, therefore, be sufficient to resist the shearing action alone, otherwise there will be danger of the bearing stress of the concrete against the rods becoming so intense as to cause the rods to work their way in and open the beam to the risk of corrosion of the steel.

The rivets between steel flanges may be looked upon somewhat like nails between timber planks, but they are in a much better condition owing to the tension induced in the rivets by contraction stresses holding the plates together, which also induces considerable friction between the plates.

As regards grip or adhesion, this is a subject certainly allied to shear, but it entails so much consideration that in addition to foregoing references the author will only refer to two important contributions to the subject—namely, a paper by William Scott Fry in the *Proceedings of the American Society of Civil Engineers*, vol. xxxvi, No. 8, October, 1910, and the *University of Illinois Bulletin*, vol. xi, No. 15, recording tests and a discussion by Mr. Duff A. Abrams.

There are many other aspects in connection with the subject of shear, as was said in the introductory remarks to the paper. Though a good deal of ground has already been covered, there is any amount of room for continued investigation. It should be borne in mind that the theoretical analysis of the stresses in materials is in the nature of a speculation; it is an attempt, like all scientific theory, to explain the facts, but with the progress of investigation further facts come to light which cause us to modify the former theory. In that way there is a gradual improvement and greater exactitude about theories. But their chief purpose must always remain to draw the attention of the practical engineer to what is happening, likely to happen, or what may possibly happen, and in practical design the complication of refinements in theory is too great to permit of adoption. The practical engineer by study of the theory will, however, be able

to make his own simplifying assumptions for approximate calculations, which will enable an economical and safe structure to be erected satisfactory in all the directions indicated by elaborate analysis.

In conclusion, the author wishes to acknowledge his indebtedness to Mr. Allan Graham, A.R.I.B.A., M.C.I., for very kindly devoting a great deal of able thought, time, and perseverance in preparing most of the diagrams for this paper.

APPENDIX I.

RESOLUTION OF A FORCE INTO ITS RECTANGULAR COMPONENTS.

In Fig. 24 the force F acting at the point O is represented by O E, and its components on the x, y, and z axes are respectively F_1, F_2, and F_3 represented by O A, O B, and O C.

Let
$$\alpha = \angle x O E$$
$$\beta_1 = \angle y O E$$
$$\gamma_1 = \angle z O E$$

be the angles which the line of action of the force makes with the axes.

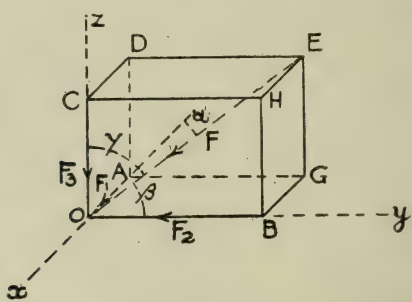


FIG. 24.

Now from the properties of right-angled triangles we have—

$$\overline{OE}^2 = \overline{OA}^2 + \overline{AE}^2 = \overline{OA}^2 + \overline{AG}^2 + \overline{EG}^2$$

which, as $AG = OB$ and $EG = OC$ becomes

$$\overline{OE}^2 = \overline{OA}^2 + \overline{OB}^2 + \overline{OC}^2$$

or
$$F^2 = F_1{}^2 + F_2{}^2 + F_3{}^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

But
$$\left.\begin{aligned} F_1 &= F \cos \alpha_1 \\ F_2 &= F \cos \beta_1 \\ F_3 &= F \cos \gamma_1 \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Therefore by substitution in (1)—

$$\cos^2 \alpha_1 + \cos^2 \beta_1 + \cos^2 \gamma_1 = 1 \quad . \quad . \quad . \quad . \quad (3)$$

These cosines are called the direction cosines and are usually symbolized by the letters l, m, and n.

Thus (3) becomes—

$$l^2 + m^2 + n^2 = 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

APPENDIX II.

CONJUGATE STRESSES.

In Fig. 25 let A B C D represent in section a prismatic element of a body surrounding the point O, bounded by the planes A D and B C parallel to each other, and parallel to a plane represented in section by y O y which passes through the point O, and bounded by the planes A B and D C which are likewise parallel to each other and parallel to the plane

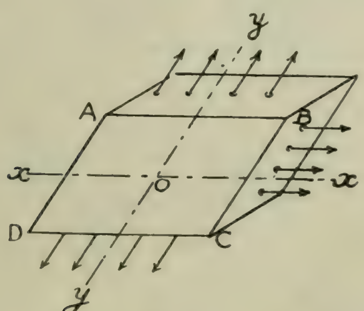


FIG. 25.

represented in section by x O x, which similarly passes through the point O. Let the stress on the plane x O x be in the direction y O y. The equal resultant forces exerted by the other parts of the material on the faces A B and D C of the parallelopiped are directly opposed and their common line of action passes through the point O; they are thus independently balanced. Now the forces exerted by the other parts of the material on the faces A D and B C must be independently balanced and have their resultants directly opposed, because there is a state of

equilibrium existing. This cannot be unless the direction of the forces is parallel to the plane $x \mathrm{O} x$. Consequently if the stress on a given plane in a body be in a given direction, the stress on any plane parallel to that direction must be in a direction parallel to the first-mentioned plane. Such pairs of stresses are termed "conjugate."

Three is the greatest number of conjugate stresses that can exist at any point, for it is obviously impossible to have a fourth stress which shall be conjugate at once to each of the other three.

If all three conjugate stresses be perpendicular to each other the normals to the planes of action of those stresses will be perpendicular to each other and will coincide with the direction of those stresses.

APPENDIX III.

Planes of Equal Shear.

Let Fig. 26 represent a parallelopiped element whose front face is the plane of the paper and whose top and bottom faces are subject to tangential stress s_1. For equilibrium against motion of translation the forces on the faces are equal, forming a couple.

Calling the area of each face $A_1 = b\,l$ we have as the moment of the couple $s_1 A_1 a_1$, where a_1 is the perpendicular distance of faces from each other.

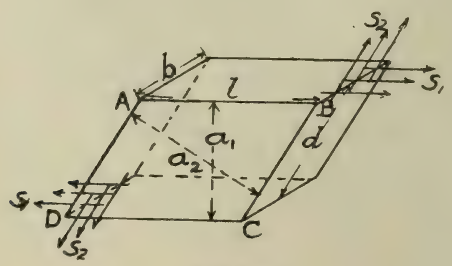


Fig. 26.

The condition of equilibrium against motion of rotation requires the application of an opposite couple with an equal moment to that of the first couple. Let the tangential stress on the other faces be s_2 and the area of each face $A_2 = b\,d$. The moment will be $s_2 A_2 a_2$ where a_2 is the perpendicular distance of the faces from each other.

Then $$s_1 A_1 a_1 = s_2 A_2 a_2$$

But both $A_1 a_1$ and $A_2 a_2$ alike represent the volume of the parallelopiped. Thus we get—

$$s_1 = s_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

or the stress on the plane A B is accompanied by an equal stress on plane A D.

This means that if the stresses on a given pair of planes are tangential to those planes and are parallel to a third plane which is perpendicular to the pair of planes aforesaid (this third plane in example is the plane of the paper), the tangential stresses must be of equal intensity.

APPENDIX IV.

Stress Components.

Taking a cube as our infinitesimal element we can denote the forces applied to the faces of the cube (no matter what those forces may be) by their rectangular components, which are respectively normal to each

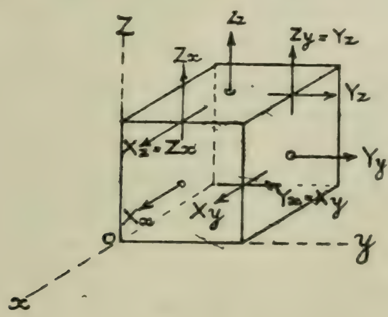


Fig. 27.

face and parallel to its edges, There are thus three possible components for each face, which gives us eighteen in all for the six faces of the cube. For equilibrium the forces directly opposed to each other on opposite faces must be equal. Therefore the total number of differing values becomes reduced from eighteen to fifteen.

Again, the conditions of equilibrium against motion of translation require the parallel tangential components on opposite faces to be equal and opposite so as to form couples, and this reduces the fifteen different components to nine, as there are two tangential forces along each of the three pairs of faces.

Further, we have from Appendix III that the tangential forces in the same plane are equal on adjoining faces, so that the nine components become reduced to six finally.

Thus to specify the stress completely at any point only six components, three normal and three tangential are required in all.

Fig. 27 will help to make this clearer. The nine component forces are shown as—

$$\begin{matrix} X_x, X_y, X_z \\ Y_x, Y_y, Y_z \\ Z_x, Z_y, Z_z \end{matrix}$$

The principle of this notation is that the large letters denote the direction of the forces, while the subscripts give the direction of the normals to the faces on which those forces act.

As
$$\begin{aligned} Z_y &= Y_z \\ X_z &= Z_x \\ Y_x &= X_y \end{aligned}$$

we get left with three normal component tractions, X_x, Y_y, Z_z, and three tangential component tractions, Y_z, Z_x, and X_y.

It is next required to establish certain relations between the sides of a rectangular pyramid as drawn in Fig. 28.

$O\,x$, $O\,y$, and $O\,z$ are the three axes at right angles to each other, and $A\,B\,C$ is the inclined face of the pyramid, whose other faces have the areas $A\,O\,B$, $A\,O\,C$, and $B\,O\,C$. Call these respectively A, A_1, A_2, A_3.

Let
$$\begin{aligned} A\,O &= a \\ B\,O &= b \\ C\,O &= c \end{aligned}$$

Let $O\,N$ be the normal to the inclined face and call it n. Let it make the angles α, β, and γ with the x, y, and z axes respectively.

Then
$$n = a \cos \alpha = b \cos \beta = c \cos \gamma \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Now the volume of a pyramid is equal to the area of its base multiplied by one-third its perpendicular height. This can be expressed in the present case by either of the following :—

$$\tfrac{1}{3} A\, n = \tfrac{1}{3} A_1\, c = \tfrac{1}{3} A_2\, b = \tfrac{1}{3} A_3\, a$$

Substituting for n the values given by Equation (6) we derive—

$$\left.\begin{aligned} A_1 &= A \,.\, \cos.\,\gamma \\ A_2 &= A \,.\, \cos.\,\beta \\ A_3 &= A \,;\, \cos.\,\alpha \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (7)$$

Knowing the directions and intensities of the six component stresses, which, as has been said, specify the stress completely at any point, it is now required to find the direction and intensity on any other plane inclined to the rectangular ones.

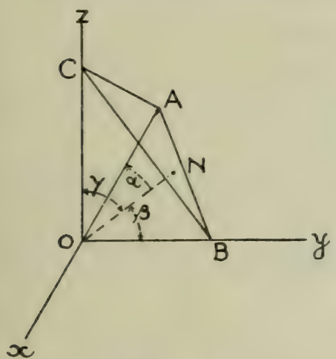


FIG. 28.

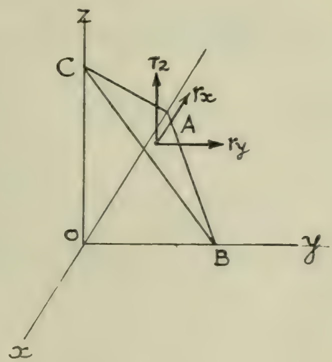


FIG. 29.

In Fig. 29 let A B C be the plane upon which the resultant stress is required. Call its area A, while the areas of the faces of the pyramid of which it forms a part we will, as before, call A_1, A_2, and A_3 respectively.

Now as the forces on the faces of the pyramid are in equilibrium the components of the reaction on the area A must each equal the forces on the faces A_1, A_2, and A_3 in the same direction. Let r_x be the resultant stress in the direction of the x axis, and r_y and r_z the resultant stresses likewise in the direction of the y and x axes respectively. Then the total components are Ar_x, Ar_y, and Ar_z respectively, which can be equated thus—

$$\left.\begin{aligned} Ar_x &= Z_x A_1 + X_y A_2 + X_x A_3 \\ Ar_y &= Y_z A_1 + Y_y A_2 + X_y A_3 \\ Ar_z &= Z_z A_1 + Y_z A_2 + Z_x A_3 \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \qquad (8)$$

Substituting the values of A_1, A_2, and A_3 from (6) we get—

$$\left.\begin{aligned} A\,r_x &= Z_x A\cos\gamma + X_y A\cos\beta + X_x A\cos\alpha \\ A\,r_y &= Y_z A\cos\gamma + Y_y A\cos\beta + X_y A\cos\alpha \\ A\,r_z &= Z_z A\cos\gamma + Y_z A\cos\beta + Z_x A\cos\alpha \end{aligned}\right\} \quad . \quad . \quad . \quad (9)$$

Of course the A's cancel out.

Regarding the stresses as forces we have by Equation (1) of Appendix I the relation—

$$r^2 = r_x^{\,2} + r_y^{\,2} + r_z^{\,2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Also, similarly to Equation (2), Appendix I—

$$\begin{aligned} r_x &= r\cos\alpha_1 \\ r_y &= r\cos\beta_1 \\ r_z &= r\cos\gamma_1 \end{aligned}$$

i.e. the direction of the stress is given by the following direction cosines—

$$\left.\begin{aligned} \cos\alpha_1 &= \frac{r_x}{r} \\ \cos\beta_1 &= \frac{r_y}{r} \\ \cos\gamma_1 &= \frac{r_z}{r} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

Supposing now we set the condition that the direction of stress shall coincide with the normal to the plane. Then we have the identities from (6) and (11) that—

$$\begin{aligned} \frac{r_x}{r} &= \cos\alpha_1 = \cos\alpha \\ \frac{r_y}{r} &= \cos\beta_1 = \cos\beta \\ \frac{r_z}{r} &= \cos\gamma_1 = \cos\gamma \end{aligned}$$

Inserting these in Equation (9) and cancelling A we get—

$$\left.\begin{aligned} (X_x - r)\cos\alpha + X_y\cos\beta + Z_x\cos\gamma &= 0 \\ X_y\cos\alpha + (Y_y - r)\cos\beta + Y_z\cos\gamma &= 0 \\ Z_x\cos\alpha + Y_z\cos\beta + (Z_z - r)\cos\gamma &= 0 \end{aligned}\right\} \quad . \quad . \quad . \quad (12)$$

From the second of these—

$$\cos \alpha = \frac{(r - Y_y) \cos \beta - Y_z \cos \gamma}{X_y}$$

Inserting in the first Equation (12) we get—

$$\frac{(X_x - r)}{X_y}\left[(r - Y_y) \cos \beta - Y_z \cos \gamma\right] + X_y \cos \beta + Z_x \cos \gamma = 0.$$

or $(X_x - r)\left[(r - Y_y) \cos \beta - Y_z \cos \gamma\right] + X_y^2 \cos \beta$

$$+ X_y Z_x \cos \gamma = 0,$$

from which $$\cos \gamma = \frac{\left[(X_x - r)(Y_y - r) - X_y^2\right]}{X_y Z_x - Y_z (X_x - r)} \cos \beta \quad . \quad . \quad (13)$$

Inserting in the third equation (12) we get—

$$\frac{Z_x}{X_y}\left[(r - Y_y) \cos \beta - Y_z \cos \gamma\right] + Y_z \cos \beta + (Z_z - r) \cos \gamma = 0.$$

or—

$$Z_x (r - Y_y) \cos \beta - Y_z Z_x \cos \gamma + X_y Y_z \cos \beta$$

$$+ X_y (Z_z - r) \cos \gamma = 0$$

from which $$\cos \gamma = \frac{\left[Z_x (Y_y - r) - X_y Y_z\right] \cos \beta}{X_y (Z_z - r) - Y_z Z_x} \quad . \quad . \quad . \quad (14)$$

Equating (13) and (14) we have—

$$\frac{(X_x - r)(Y_y - r) - X_y^2}{X_y Z_x - Y_z (X_x - r)} = \frac{Z_x (Y_y - r) - X_y Y_z}{X_y (Z_z - r) - Y_z Z_x}$$

From which—

$$\left(X_x Y_y - r Y_y - r X_x + r^2 - X_y^2\right)\left(Z_z - r - \frac{Y_z Z_x}{X_y}\right)$$

$$- \left(Z_x - \frac{X_x Y_z}{X_y} + \frac{r Y_z}{X_y}\right)\left(Y_y Z_x - r Z_x - X_y Y_z\right) = 0$$

This gives—

$$+r^3-r^2(Z_z+Y_y+X_x)+r(Y_y Z_z+X_x Z_z+X_x Y_y$$
$$-X_y{}^2-Z_x{}^2-Y_z{}^2)-X_x Y_y Z_z-2 X_y Y_z Z_x$$
$$+X_y{}^2 Z_z+Y_y Z_x{}^2+X_x Y_z{}^2=0 \quad . \quad . \quad . \quad (15)$$

Calling $$Z_z+Y_y+X_x=C_1$$

$$Y_y Z_z+X_x Z_z+X_x Y_y-X_y{}^2-Z_x{}^2-Y_z{}^2=C_2$$

and—

$$X_x Y_y Z_z+2 X_y Y_z Z_x-X_y{}^2 Z_z-Y_y Z_x{}^2-X_x Y_z{}^2=C_3$$

we can express (15) as an ordinary cubic equation—

$$r^3-C_1 r^2+C_2 r-C_3=0$$

which of course is known to have three roots or values for the stress r, which thus satisfy the condition of being normal to their planes of action; and according to the properties of conjugate stresses referred to in the last paragraph of Appendix II we see that the directions of the three normal stresses must be perpendicular to each other.

APPENDIX V.

Ordinary Mathematical Analysis of Stresses and Strains.

The following illustrates the manner of resolving stresses into their components tangential and normal to any plane.

Let Fig. 30 represent a bar subjected to a simple pull P normal to the plane A B C D.

Calling p the stress at any point O, it is required to find the component stresses on the plane E F G H inclined at angle θ. Drawing the infinitesimal prism in Fig. 31, we see that the area of the inclined face is A B C D . sec θ (see Fig. 32). The force on A B C D is $P=p$. (A B C D), which resolved normal to E F G H gives $P\cos\theta$ (see Fig. 33).

Dividing by the larger surface, we get the normal component stress—

$$p_n=\frac{P\cos\theta}{\text{A B C D}\,.\,\sec\theta}=\frac{p\,.\,\text{A B C D}\cos\theta}{\text{A B C D}\sec\theta}=p\cos^2\theta$$

Similarly the tangential component stress (see Fig. 33) is—

$$p_t = \frac{P \sin\theta}{A\,B\,C\,D\,.\sec\theta} = \frac{p\,.\sin\theta}{\sec\theta} = p \sin\theta\cos\theta$$

or

$$p_t = \frac{p}{2}\,.\sin 2\,\theta$$

It is evident that p_t reaches the maximum value of $\frac{p}{2}$ when $\theta = 45°$, so that surfaces inclined at 45° to the surface upon which the stress is wholly normal are subjected to maximum shear stress which amounts to half the intensity of the original stress.

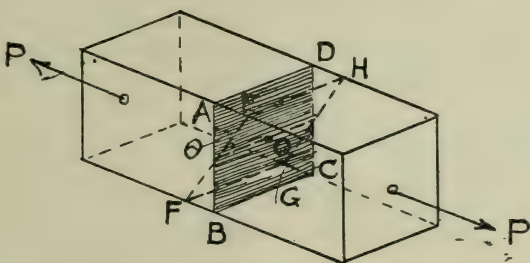


FIG. 30.

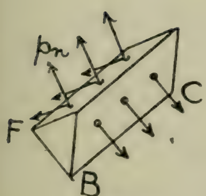


FIG. 31.

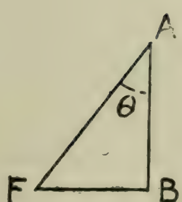


FIG. 32.

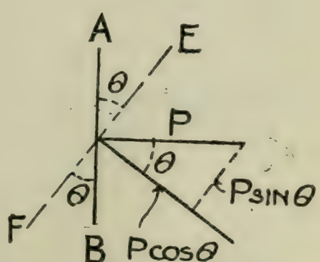


FIG. 33.

Now let us consider the combination of two known conjugate stresses normal to two planes at right angles to each other.

If we wish to find the stress across any oblique plane perpendicular to the plane of the paper across which there is no stress, let p_x and p_y be the given stresses normal to two perpendicular planes, A B and C D. As p_x and p_y may vary, we will consider the equilibrium of an indefinitely small portion, which is drawn to a large scale in Fig. 34, in which the

analysis will be applied to unit thickness perpendicular to the plane of the paper. Let the inclined plane make an angle θ to the mutually perpendicular planes so that the normal O N is inclined θ to O X, $\left(\frac{\pi}{2}-\theta\right)$ to O Y. The stresses p_x and p_y are shown alike in the figure, but merely a slight alteration of the algebra will be required for unlike stresses.

The total force normal to the elemental face E F is $P_x = p_x \times EF$, the area being $EF \times$ unity. The total force on F G normal to the face is $P_y = p_y \times FG$.

Let p_n and p_t be the normal and tangential stress intensities respectively on the face E G which will be considered positive in the directions O N and O E. Now consider the equilibrium of the wedge E F G. Resolving forces in direction O N, we have—

$$\begin{aligned} p_n \times EG &= P_x \cos\theta + P_y \cos\left(\frac{\pi}{2}-\theta\right) \\ &= p_x \,.\, EF \,.\, \cos\theta + p_y \,.\, FG \,.\, \sin\theta \\ &= p_x \,.\, EG \,.\, \cos\theta \,.\, \cos\theta + p_y \,.\, EG \,.\, \sin\theta \,.\, \sin\theta \end{aligned}$$

Dividing by E G—

$$p_n = p_x \cos^2\theta + p_y \sin^2\theta \quad . \quad . \quad . \quad . \quad (16)$$

Resolving in direction O E—

$$\begin{aligned} p_t \times EG &= P_x \sin\theta - P_y \cos\theta \\ &= p_x \,.\, EF \,.\, \sin\theta - p_y \,.\, FG \,.\, \cos\theta \end{aligned}$$

Expressing E F and F G in terms of E G and dividing by E G—

$$p_t = (p_x - p_y)\sin\theta\cos\theta = \frac{p_x - p_y}{2}\sin 2\theta \quad . \quad . \quad (17)$$

If $\theta = 45°$, the shear stress intensity

$$p_t = \frac{p_x - p_y}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

which is a maximum.

Across the same plane at 45° the direct stress intensity (in this case tension) from Equation (16) is—

$$p_n = p_x \cos^2 45^\circ + p_y \sin^2 45^\circ = \frac{p_x + p_y}{2}$$

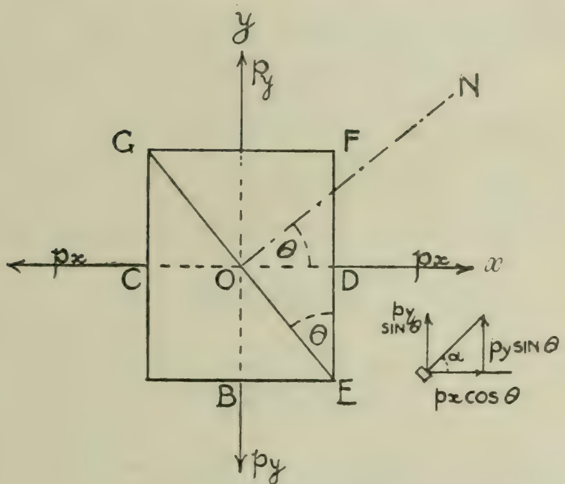


FIG. 34. FIG. 35.

Combining (16) and (17), if p is the intensity of the resultant stress, since the two forces P_x and P_y are equal to the rectangular components of the force $p \times EG$,

$$P = \sqrt{P_x^{\,2} + P_y^{\,2}}$$

$$p \,.\, EG = \sqrt{(p_x \,.\, EF)^2 + (p_y \,.\, FG)^2}$$

$$= EG \sqrt{p_x^{\,2} \cos^2 \theta + p_y^{\,2} \sin^2 \theta}$$

$$p = \sqrt{p_x^{\,2} \cos^2 \theta + p_y^{\,2} \sin^2 \theta}$$

Now $\sin^2 \theta + \cos^2 \theta = 1$, therefore—

$$p_x^2 \cos^2\theta + p_y^2 \sin^2\theta$$
$$= p_x^2 \cos^2\theta\,(\sin^2\theta + \cos^2\theta) + p_y^2 \sin^2\theta\,(\sin^2\theta + \cos^2\theta)$$
$$= p_x^2 \cos^4\theta + p_y^2 \sin^4\theta + \sin^2\theta \cos^2\theta\,(p_x^2 + p_y^2)$$
$$= p_x^2 \cos^4\theta + p_y^2 \sin^4\theta + 2\,p_x p_y \sin^2\theta \cos^2\theta$$
$$+ \sin^2\theta \cos^2\theta\,(p_x^2 + p_y^2) - 2\,p_x p_y\,(\sin^2\theta \cos^2\theta)$$
$$= (p_x \cos^2\theta + p_y \sin^2\theta)^2 + \sin^2\theta \cos^2\theta\,(p_x^2 + p_y^2 - 2\,p_x p_y)$$
$$= pn^2 + (p_x - p_y)^2 \sin^2\theta \cos^2\theta = p_n^2 + p_t^2$$

Inserting this in the foregoing equation we obtain—

$$p = \sqrt{p_n^2 + p_t^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

Now $\qquad P_y = p_y\,FG = p_y\,EG \sin\theta$

and $\qquad P_x = p_x\,EF = p_x\,EG \cos\theta$

Dividing both by E G to get the intensity of stress we find the component stresses $p_x \cos\theta$ and $p_y \sin\theta$ in directions Ox and Oy respectively.

Now from Fig. 35 we see that

$$\tan\alpha = \frac{p_y \sin\theta}{p_x \cos\theta} = \frac{p_y}{p_x} \tan\theta \quad . \quad . \quad . \quad . \quad . \quad (20)$$

and p, the resultant stress, makes an angle ϕ with the normal to the plane E G across which it acts, such that

$$\tan\phi = \frac{p_t}{p_n} = \frac{(p_x - p_y)\sin\theta\cos\theta}{p_x \cos^2\theta + p_y \sin^2\theta} \quad . \quad . \quad . \quad . \quad (21)$$

To find the plane across which the resultant stress is most inclined to the normal we proceed as follows :—

When ϕ is a maximum, $\tan\phi$ is a maximum, so that

$$\frac{d\,(\tan\phi)}{d\,\theta} = 0$$

Therefore we differentiate and equate to zero, obtaining—

$$(p_x - p_y) \sin\theta \cos\theta \,.\, \sin 2\theta + (p_x \cos^2\theta + p_y \sin^2\theta) \cos 2\theta = 0$$

in which we insert values from (16) and (17) so as to obtain—

$$p_n \cos 2\theta + p_t \sin 2\theta = 0$$

or
$$\tan 2\theta = -\frac{p_n}{p_t} = -\cot\phi = \tan\left(\frac{\pi}{2} + \phi\right)$$

(Compare Formula (21).)

Therefore
$$2\theta = \frac{\pi}{2} + \phi$$

and
$$\theta = \frac{\pi}{4} + \frac{\phi}{2}$$

Putting Equation (21) in the form—

$$\tan\phi = \frac{(p_x - p_y)\sin 2\theta}{p_x(1 + \cos 2\theta) + p_y(1 - \cos 2\theta)}$$

and remembering that—

$$\sin\left(\frac{\pi}{2} + \phi\right) = \cos\phi, \text{ while } \cos\left(\frac{\pi}{2} + \phi\right) = -\sin\phi$$

we get by substitution—

$$\tan\phi = \frac{(p_x - p_y)\cos\phi}{p_x(1 - \sin\phi) + p_y(1 + \sin\phi)}$$

from which
$$\frac{p_y}{p_x} = \frac{1 - \sin\phi}{1 + \sin\phi} \quad \ldots \ldots \quad (22)$$

or
$$\sin\phi = \frac{p_x - p_y}{p_x + p_y} \quad \ldots \ldots \quad (23)$$

Equation (23) gives the maximum inclination to the normal, while Equation (22) is used in the determination of the stresses in granular substances.

When the two given stresses p_x and p_y are unlike, as, for instance, p_x is tensile and p_y is compressive, as in Fig. 36, the components are—

$$p_n = p_x \cos^2\theta - p_y \sin^2\theta \text{ (tensile)} \quad \ldots \ldots \quad (16a)$$

$$p_t = (p_x + p_y)\sin\theta\cos\theta = \tfrac{1}{2}(p_x + p_y)\sin^2\theta \quad \ldots \quad (17a)$$

The maximum shear, just as before, occurs when $\theta = 45°$, its value being—

$$p_t = \frac{p_x + p_y}{2} \quad \ldots \ldots \quad (18a)$$

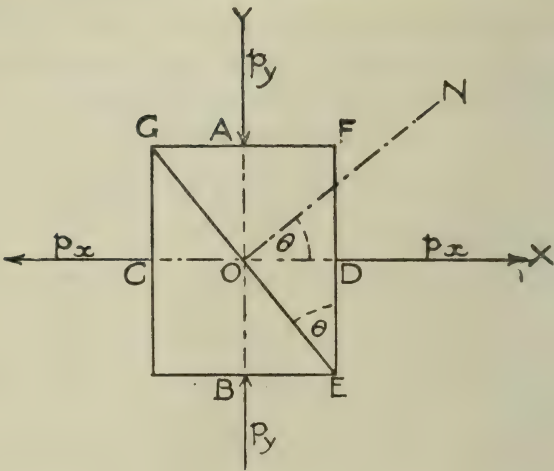


FIG. 36.

In the special case of unlike stresses, where p_x and p_y are of equal intensity, the values for $\theta = 45°$ are—

$$p_t = \frac{p_x + p_y}{2} = p_x = p_y$$

$$p_n = 0$$

This exactly corresponds with the case of pure shear.

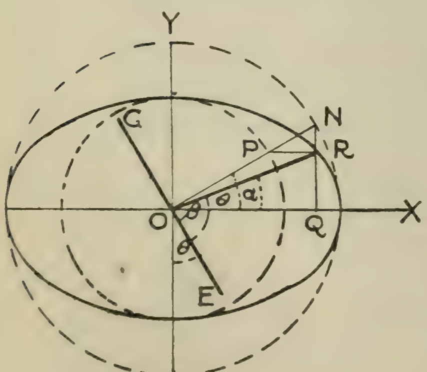


FIG. 37.

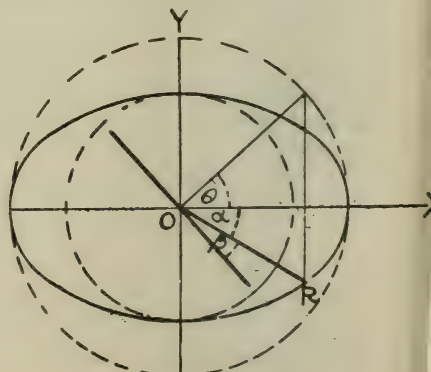


FIG. 38.

A graphical construction, generally termed the "Ellipse of Stress," by which the resultant stress can be obtained is as follows :—

Referring to Fig. 37, with O as centre describe the larger and smaller dotted circles with radii proportional to p_x and p_y respectively. Draw O N normal to the plane E G to meet the larger circle in N and the smaller in P. Draw N Q perpendicular to O X and P R perpendicular to O Y to meet N Q in R. Then O R represents the resultant stress p both in direction and magnitude of intensity.

The locus of R for various values of θ, *i.e.* for different oblique planes, is evidently an ellipse, for the co-ordinate O Q along O X is—

$$\text{O N} \cos\theta \text{ or } p_x \cos\theta$$

and R N the co-ordinate along O Y is—

$$\text{O P} \sin\theta \text{ or } p_y \sin\theta$$

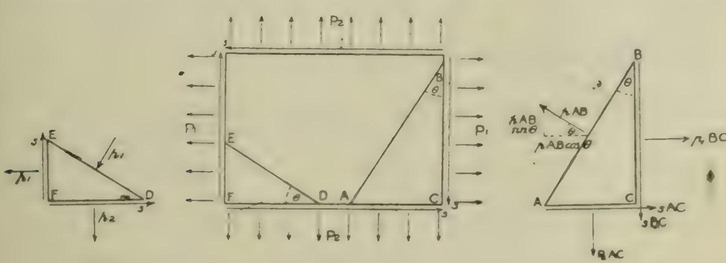
The axes of the ellipse are the axes of stress. Also it is obvious from the figure that

$$\tan\alpha = \frac{p_y \sin\theta}{p_x \cos\theta} = \frac{p_y}{p_x} \tan\theta$$

In the second case, where, say, p_y is negative and p_x is positive, O R (Fig. 38) will represent the resultant stress in and magnitude of intensity. It will be noticed that in this case $\tan\alpha$ is negative and β is obviously less than β in Fig. 37.

We may now proceed to find principal stresses and planes in a few simple, two-dimensional cases where the stress perpendicular to the plane of the paper is nil.

Referring to Fig. 39, showing the elevation of an indefinitely small rectangular parallelopiped of unit thickness, let there be on the mutually



(*c*) (*a*) (*b*)

FIG. 39.

perpendicular planes, normal stresses, one of intensity p_1 and the other of intensity p_2, in addition to the two equal shear stresses of intensity s. These stresses, p_1, p_2, and s may be considered either as stresses resulting directly from different kinds of external action, or as rectangular component stresses, normal and tangential, into which oblique stresses, on the faces perpendicular to the figure, have been resolved, as was explained previously.

Suppose, now, that the direction of the principal planes and the intensity of the principal stresses upon them be as shown by A B and E D in Fig. 39 (*a*), θ being the angle of inclination as marked thereon.

Considering the equilibrium of the element A B C, Fig. 39 (*b*) will represent the forces acting on its faces.

Resolving forces parallel to A C—

$$p \,.\, \overline{AB} \times \cos\theta = p_1 \,.\, \overline{BC} + s \,.\, \overline{AC}$$

$$= p_1 \,.\, \overline{AB}\cos\theta + s \,.\, \overline{AB}\sin\theta$$

hence
$$(p - p_1)\cos\theta = s\sin\theta$$

$$p - p_1 = s\tan\theta \qquad \ldots \ldots \qquad (24)$$

Resolving parallel to B C—

$$p \,.\, \overline{AB} \times \sin\theta = p_2 \,.\, \overline{AC} + s \,.\, \overline{BC}$$

$$= p_2 \,.\, \overline{AB}\sin\theta + s \,.\, \overline{AB}\cos\theta$$

hence
$$(p - p_2)\sin\theta = s\cos\theta$$

$$p - p_2 = s\cot\theta \qquad \ldots \ldots \qquad (25)$$

Subtracting Equation (24) from Equation (25)—

$$p_1 - p_2 = s(\cot\theta - \tan\theta) = \frac{2s}{\tan 2\theta}$$

$$\tan 2\theta = \frac{2s}{p_1 - p_2} = -\frac{2s}{p_2 - p_1} \qquad \ldots \qquad (26)$$

Since $\tan(180° - A) = -\tan A$, we can find two values of θ differing by a right angle, which means that the inclinations to B C of the two principal planes are mutually perpendicular.

Now, multiplying (24) by (25)—

$$(p-p_1)(p-p_2)=s^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

$$p^2-p(p_1+p_2)-(s^2-p_1 p_2)=0$$

$$p=\tfrac{1}{2}(p_1+p_2)\pm\sqrt{\tfrac{1}{4}(p_1+p_2)^2+(s^2-p_1 p_2)} \quad . \quad . \quad (28)$$

$$=\tfrac{1}{2}(p_1+p_2)\pm\sqrt{\tfrac{1}{4}(p_1-p_2)^2+s^2}$$

These two values of p are the values of the stress intensities on the two principal planes. The larger value (where the upper sign is taken) will be the stress intensity on such a plane as A B (Fig. 39(*a*)), and will be of the same sign as p_1 and p_2; the smaller value, say p', will be that on such a plane as E D (Fig. 39(*a*)) perpendicular to A B, and will be of opposite sign to p_1 and p_2 if s^2 is greater than $p_1 p_2$.

The planes on which there are maximum shear stresses are inclined 45° to the principal planes found, and the maximum intensity of shear stress is by Equation (18) or (18*a*)—

$$s_{\text{max.}}=\frac{p-p'}{2}=\sqrt{\tfrac{1}{4}(p_1+p_2)^2+s^2-p_1 p_2}=\sqrt{\tfrac{1}{4}(p_1-p_2)^2+s}$$

It is easy in the foregoing to adjust the formula for the case where p_1 or p_2 is of negative sign, or where p_1 or p_2 is zero. As special use is made of the result when p_1 or p_2 is zero, it is worth while working out the result in detail by the foregoing method.

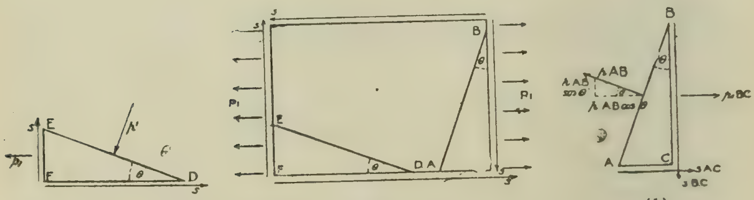
Thus Fig. 40 shows the forces on an indefinitely small rectangular element of unit thickness, θ being the inclination of a principal plane A B to the plane B C, which has normal stress of intensity p_1, and a shear stress of intensity s acting on it. Let p be the intensity of the wholly normal stress on A B. The face F C has only the shear stress of intensity s acting tangentially to it.

Consider the equilibrium of the wedge A B C; resolving the forces parallel to A C (Fig. 40 (*b*))—

$$p \cdot \overline{AB} \cdot \cos\theta = p_1 \cdot \overline{BC} \times s \cdot \overline{AC}$$
$$= p_1 \overline{AB} \cdot \cos\theta + s \cdot \overline{AB} \sin\theta$$

$$(p - p_1)\cos\theta = s\sin\theta$$

$$(p - p_1) = s\tan\theta \quad \ldots\ldots\ldots \quad (29)$$



(c) (a) (b)

FIG. 40.

Resolving parallel to B C—

$$p \cdot \overline{AB} \cdot \sin\theta = s \cdot \overline{BC} = s \cdot \overline{AB}\cos\theta$$

$$\tan\theta = \frac{s}{p} \quad \ldots\ldots\ldots \quad (30)$$

Substituting for $\tan\theta$ in (29)—

$$(p - p_1) = \frac{s^2}{p}$$

$$p^2 - p_1 p - s^2 = 0$$

$$p = \tfrac{1}{2}p_1 \pm \sqrt{\tfrac{1}{4}p_1{}^2 + s^2} = \frac{p_1}{2}\left(1 \pm \sqrt{1 + \frac{4s^2}{p_1{}^2}}\right) \quad . \quad (31)$$

and the values of θ may be found by substituting these values of p in (30).

For instance—

$$\tan\theta = \frac{s}{\tfrac{1}{2}p_1\left(1 + \sqrt{1 + \frac{4s^2}{p_1{}^2}}\right)} \text{ or } \frac{s}{\tfrac{1}{2}p_1\left(1 - \sqrt{1 + \frac{4s^2}{p_1{}^2}}\right)} \quad (32)$$

which is of the form—

$$\frac{s}{C(1+K)} \text{ or } \frac{s}{C(1-K)} = \frac{K'}{1+K} \text{ or } \frac{K'}{1-K}$$

Therefore—

$$K' = \tan\theta\,(1 + K) \text{ or } \tan\theta\,(1 - K)$$

The two values differ by a right angle, the principal planes being at right angles. A B (Fig. 40 (*b*)) shows a principal plane of greatest stress corresponding to—

$$p = \tfrac{1}{2}p_1 + \sqrt{\tfrac{1}{4}p_1^2 + s^2} = \frac{p_1}{2}\left(1 + \sqrt{1 + \frac{4s^2}{p_1^2}}\right)$$

and E D (Fig. 40 (*c*)) shows the other principal plane on which the normal stress is—

$$p' = \tfrac{1}{2}p_1 - \sqrt{\tfrac{1}{4}p_1^2 + s^2} = \frac{p_1}{2}\left(1 - \sqrt{1 + \frac{4s^2}{p_1^2}}\right)$$

of opposite sign to p_1.

The planes of greatest shear stress are inclined at 45° to the principal planes, and the intensity of shear stress upon them is—

$$s_{max.} = \frac{p - p'}{2} = \sqrt{\tfrac{1}{4}p_1^2 + s^2} = \frac{p_1}{2}\sqrt{1 + \frac{4s^2}{p_1^2}} \quad . \quad . \quad (33)$$

If an elastic material be subjected to a stress it will undergo deformation. A tension stress f of unit intensity acting alone will produce a unit deformation or strain σ in its own direction.

This longitudinal strain is proportional to the stress, or $f \propto \sigma$.

Let $\sigma_l = \frac{f}{E}$ where E is a function called Young's modulus of elasticity or the stretch modulus. This function varies for different materials. For steel it is fairly uniform and constant within elastic limits, but for concrete it is a variable. Accompanying the strain in the axis of the stress there will be contraction in all directions. The amount of this transverse contraction again differs for different materials, and varies as the longitudinal strain.

Let the transverse strain $\sigma_t = \Pi\sigma_l$. Therefore $\sigma = \Pi\frac{f}{E}$ where Π is Poisson's ratio.

An isotropic material has the same elastic properties in all directions, and the effect upon such a material of a stress f_2 acting alone at right

angles to the direction of f_1 would be to produce a strain in its own direction, σ_2, such that—

$$\sigma_2 = \frac{f_2}{E}$$

and at right angles to this, including the direction of the strain $\frac{f_1}{E}$, a contraction strain—

$$\frac{\Pi f_2}{E}$$

Similarly a stress f_3, the direction of which is perpendicular to both the previously mentioned stresses, will produce in addition to its longitudinal strain a contraction strain—

$$\frac{\Pi f_3}{E}$$

in all directions perpendicular to its direction, including the direction of the stress f_1.

If at a point in isotropic material there are three principal stresses of intensities f_1, f_2, and f_3, each will independently produce the same strains which it would cause if acting alone. If the stresses be all of the same sign the total strain produced in the direction of the stress f_1 will then be—

$$\sigma_1 = \frac{f_1}{E} - \frac{\Pi}{E}(f_2 + f_3) \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

while in the direction of f_2 the strain—

$$\sigma_2 = \frac{f_2}{E} - \frac{\Pi}{E}(f_1 + f_3) \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

and in the direction of f_3 the strain—

$$\sigma_3 = \frac{f_3}{E} - \frac{\Pi}{E}(f_1 + f_2) \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

If any of the stresses be of opposite kind, the strains will be found by changing the sign of the particular stress in each of the above equations.

Let Fig. 41 represent an indefinitely small element subjected to stresses on all faces, as shown. Then the strains that will be produced in the various directions may be tabulated as follows, calling the forces f_x in direction x, and so on.

Stress.	Strain in direction of x	Strain in direction of y	Strain in direction of z
f_x	$\frac{f_x}{E}$	$-\frac{\Pi}{E}f_x$	$-\frac{\Pi}{E}f_x$
f_y	$-\frac{\Pi}{E}f_y$	$\frac{f_y}{E}$	$-\frac{\Pi}{E}f_y$
f_z	$-\frac{\Pi}{E}f_z$	$-\frac{\Pi}{E}f_z$	$\frac{f_z}{E}$

These equations define the strains in any direction due to the stresses acting alone, and if we wish to find the resulting strain from two or more acting together, this is done by adding the separate strains, due attention being paid to the signs.

Now it has been shown above that a shear stress consists of two equal stresses of opposite sign acting at right angles to one another. The resulting strain can be obtained by adding the strains given in above table due to stresses f_1 and f_2, which are of opposite sign and act at right angles to each other. The strains are—

$$\left.\begin{aligned} \frac{f_1}{E}+\frac{\Pi}{E}f_2 &= \frac{f}{E}(1+\Pi) \text{ in direction 1} \\ -\frac{f_2}{E}-\frac{\Pi}{E}f_1 &= -\frac{f}{E}(1+\Pi) \text{ in direction 2} \\ -\frac{\Pi}{E}f_1+\frac{E}{\Pi}f_2 &= 0 \text{ in direction 3} \end{aligned}\right\} \quad . \quad . \quad . \quad (37)$$

Thus the strain in the two directions has increased by Π due to the superposition of the two stresses, and has been reduced to zero in the third direction, *i.e.*—

$$\text{shear strain} = \varsigma = \frac{f}{E}(1+\Pi)$$

whereas the strain due to a direct tensile stress is only—

$$\tau = \frac{f}{E}$$

Consequently
$$\frac{\varsigma}{\tau} = \frac{\frac{f}{E}(1+\Pi)}{\frac{f}{E}} = 1 + \Pi$$

or
$$\varsigma = \tau(1+\Pi)$$

If $\Pi = \frac{1}{3}$ then $\frac{\varsigma}{\tau} = \frac{4}{3}$, and for $\Pi = \frac{1}{4}$ then $\frac{\varsigma}{\tau} = \frac{5}{4}$. That is to say, if the maximum strain the material will sustain is known, then a shear stress of

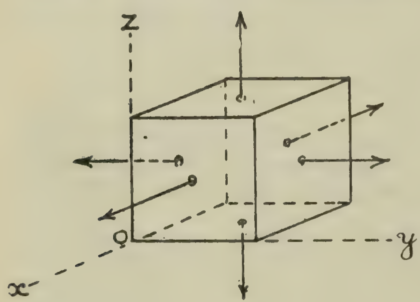


FIG. 41.

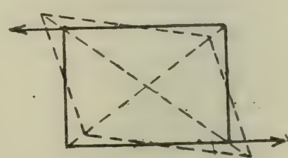


FIG. 42.

$\frac{3}{4}$ to $\frac{4}{5}$ the value of the maximum tensile stress will give the same strain. In practice the working shear stress bears the relation $\frac{4}{5}$ to the tensile stress; viz. if the latter is $7\frac{1}{2}$ tons the shear stress is taken at 6 tons.

It should be noted that Young's modulus is defined as the stretch modulus, where the strain is that produced by a tensile stress, no other stress acting with it. The action of other independent principal stresses would alter the strain produced, therefore in such cases the function E defined by the relation—

$$E = \frac{f}{\sigma}$$

would require modification. Thus the modulus should be altered if all lateral strain is prevented as at about the middle of flat slabs subjected to bending.

If strain be measured by cartesian ordinates, then if under the application of forces extension is produced of lines parallel to the x axis and of lines

parallel to the y axis without any extension or contraction parallel to the z axis, a strain will result which distorts the shape of the section. Such distortion is termed a "pure shear" strain, and is illustrated in Fig. 42. The strain is measured by angular distortion.

If after being subjected to such a pure shear strain the body be turned about its axis we have what is termed a simple shear strain. The strain is measured by angular distortion, not only on the transverse section as before, but longitudinally as to the axis of the body.

It is not proposed here to deal with the determination of the stresses in detail in sections subjected to torsion alone or combined with bending. Reference may be made thereon to Professor Love's treatise and to Professor Arthur Morley's "Strength of Materials" (Longmans, Green

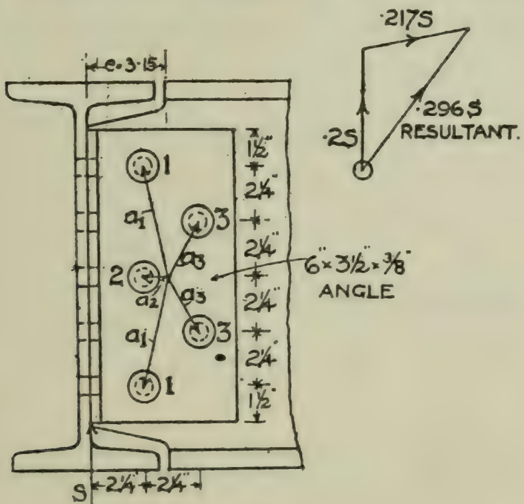


FIG. 43.

& Co.), in which it is shown that the shear stresses under a pure torsional moment on homogeneous sections are approximately as follows :—

For a square section—

$$T = 0{\cdot}208\ s\,d^3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (38)$$

For a rectangular section—

$$T = \frac{s\,b^2\,d}{3\,a + 1{\cdot}8\,b} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

where d is the longer side and b is the shorter side.

For a circular section—

$$T = \frac{\pi}{16} s\,d^3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (40)$$

In riveted work the joints are frequently subjected to eccentric loading, which results in a torsional moment on any group of rivets. Thus take for example the cleat shown in Fig. 43.

Let e be the eccentricity of the thrust, and S the reaction. The rivets have to jointly resist a torsional moment $T = S e$ and a direct thrust S. Due to inequalities in workmanship the exact distribution of stress cannot be determined, but we can proceed as follows to make an approximate calculation. The direct load taken by each rivet is $\frac{S}{5}$.

The stress induced by the torsional moment will be different in the various rivets, but its line of action will be at right angles to its arm about the centroid of the group. Therefore the forces in the rivets will be proportional to their arms. Therefore, calling the F_1 force in rivet 1, F_2 in 2, and so on, we have—

$$F_2 = F_1 \frac{a_2}{a_1}, \quad \text{and its moment will be } F_1 \cdot \frac{a_2^{\,2}}{a_1}$$

$$F_3 = F_1 \frac{a_3}{a_1}, \quad \text{and its moment } F_1 \frac{a_3^{\,2}}{a_1}$$

The total resistance moment will be—

$$R = \Sigma\, F\, a = \frac{F}{a_1} \Sigma\, a^2 = \frac{F_1}{a_1}\left(2\, a_1^{\,2} + a_2^{\,2} + 2\, a_3^{\,2}\right)$$

Equating this to the torsional moment we ascertain F_1. The direct load taken by the rivet is then combined with this torsional force, which may be done graphically or algebraically, so as to afford the direction and amount of the resultant force. A graphical combination for one rivet is shown in the figure.

APPENDIX VI.

Professor A. Rejtö's Analysis of Stresses and Strains.

Solid bodies undergo deformations by change of position of their particles and by sliding of the particles. The friction resulting from such sliding is called internal friction. If the acceleration of the sliding particles of the body is very small, the external force will have merely to overcome the internal friction. Knowing the relations between the

external force and internal friction, one can, by knowing either value determine the other. These relations are deduced as follows :—

Whenever there is deformation by reason of the exertion of an external force, an increase and a decrease of dimensions result. Consequently it may be said that every external force exerts a double action : one a *tension* which tends to cause increase in dimensions and a *compression* which tends to cause decrease in dimensions. The molecules are thus caused to slide in two directions, and the internal friction may be imagined as divided into two parts, one of which is opposed to the sliding in the direction of the increase of dimensions, and the other opposed to the sliding in the direction of the decrease of dimensions. The increase and decrease bear a certain relation to each other, so that, since the volume remains constant, one can by knowing one of the quantities determine the other. The changes of dimensions are not only in relation with each other, but also with the actions of the external force, and the latter is in relation with the internal friction, so that it is enough to consider only one of the changes of dimensions, one of the actions of the external force (the one which bears a relation to this change), and one part of the internal friction.

Professor Rejtö only takes into consideration the increase of dimension, and consequently only considers the extending action p_n and only that part of the internal friction ΦN which resists the sliding in the direction of the increase of dimensions.

Consequently it may be said that the internal friction is always opposed to the extending action of the external force, and that the friction is accordingly in equilibrium with the extending action, *i.e.*—

$$p_n = \Phi N$$

It must be added that the internal friction depends upon the form of the molecules, and, as the form of the molecules is an absolute property of matter, the internal friction will also be an absolute property of matter.

The intensity of the internal friction, *i.e.* the value of the coefficient of friction, depends upon the shape of the surfaces of contact. In the case of regular polyhedrons, and in particular for shapes almost perfectly spherical, the surfaces which will be in contact will be similar, and in such cases the internal friction, and consequently the coefficient of friction, will have a constant value. For molecules having irregular forms, altogether different surfaces will enter into contact. The projections of one molecule will penetrate into the cavities of another, and will have the effect of increasing the coefficient of friction. In such case the coefficient of friction will be variable and will prove as great as the interlocking is effective. If therefore the coefficient of friction depends upon the degree of interlocking and consequently on the relative position of neighbouring molecules, it follows that the same value ought always to be preserved for the same disposition of the

molecules, whatever may be the means by which the molecules are brought into such a position.

It may therefore be said that the coefficient of friction Φ is constant for some substances while variable for others, and that in the latter case its value will depend upon the disposition of the molecules.

Since the internal friction is an absolute property of the substance and must be considered for each force, it follows that for equal dimensions and for equal forces, the substance which will offer the greatest resistance, *i.e.* which will be the hardest, will be the one in which the internal friction has the greatest value. Therefore it can be said that the numerical value of the internal friction is the coefficient of hardness.

It follows that the substances which have a constant coefficient of friction will have a constant hardness, and those with a variable coefficient of friction will have a variable hardness, which is also augmented by some sort of deformation, so that with the former substances we must distinguish between an initial hardness $\Phi_0 N$ and a maximum hardness $\Phi_{max.} N$.

Besides the numerical value of the friction, the relation of the internal friction to the cohesion p_0 is of great importance. This relation shows us the mechanical properties (the facility, more or less considerable, of giving a definite form) of the substance, because at the time of the application of the forces it is the cohesion which transmits to the neighbouring particles the extending action of the external force. So long as the internal friction is greater than the extending action of the external force, no permanent displacement can result; but if the extending action is greater some molecules will be displaced; and if the cohesion p_0 is greater than the internal friction—

$$p_0 > \Phi N$$

it will transmit the extending action to the neighbouring molecules, *i.e.* the material will consequently be plastic (ductile).

But if the adhesion is smaller—

$$p_0 < \Phi N$$

the extension will overcome the cohesion, and the material will then undergo a rupture without deformation, *i.e.* the material will be brittle.

Taking the relation of the cohesion to the extending action and the character of the coefficient of friction, we can classify the materials in the following way.

1. Materials whose coefficient of friction Φ is constant—

If $\Phi N > p_0$ they are called brittle;

$\Phi N < p_0$ they are called plastic;

$\Phi N = p_0$ they are placed between the brittle and the plastic.

2. Materials whose coefficient of friction Φ is variable are called tenacious.

The tenacity lasts until the initial value of the hardness has reached —according to the deformation—its maximum value, $\Phi_{max.} N$.

If $\Phi_{max.} N > p_0$ the material becomes, after the tenacious period, brittle;

$\Phi_{max.} N < p_0$ the material becomes, after the tenacious period, plastic;

$\Phi_{max.} N = p_0$ the material becomes, after the tenacious period, neither brittle nor plastic.

It must also be observed that in the cases where crystalline groups are presented the coefficient of friction of the molecules for certain forces enters into consideration, for others the molecular configurations. These bodies will be placed in a third principal group, which will be called the group with the combined coefficient of friction.

As the internal friction is opposed to increase of dimensions $p_n = \Phi N$, and since for a tensile stress it is the force of tension which produces the increase of dimensions, the value of the internal friction may be most simply determined by using a tension diagram resulting from the tensile force.

In the above equation p_n and ΦN are related to a unit of section: consequently—

$$p_{nx} = \frac{P_x}{A_x} = \Phi_x N$$

$$p_{n1} = \frac{P_1}{A_1} = \Phi_1 N$$

$$p_{n2} = \frac{P_2}{A_2} = \Phi_2 N$$

In order to calculate these values we need a tension diagram representing the work of an absolutely free specimen whose molecules are not subjected to any other force than the tensile force.

Test-pieces with enlarged ends must be employed for tensile tests in order to be able to fix them in the grips of the testing machine. These enlarged ends have an influence upon the work of the bar, so that such tension diagrams as take in the lengthening of the restrained parts are wrong and cannot be used for the calculation of internal friction.

But the error resulting from the enlarged ends and the grips can be avoided if, by starting from the latter, we place two marks on the bar

about the middle of its length, at distances nearly twice the diameter apart, and if we take care that the diagram of tension only contains the lengthening of the bar comprised between the two marks.

Knowing the values of the internal friction, we may also construct graphically the diagram of the internal friction. For example, let us imagine a tenacious body whose tension diagram is represented by Fig. 44. In order to obtain a series of values of the internal work let us divide the tension diagram by perpendicular ordinates at any part, and let us determine for each division the value of Φ N and that as far as the

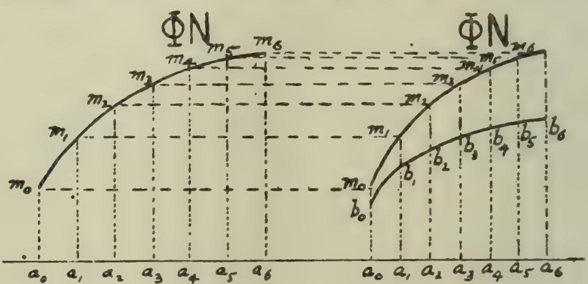


FIG. 45. FIG. 44.

maximum tensile stress, because from that point, notwithstanding the material may be plastic, $\Phi_{max.}$ N preserves a constant value.

In a_1 we have $$\Phi_1 N = \frac{P_1}{A_1} = \frac{a_1 b_1}{A_1}$$

In a_2 $$\Phi_2 N = \frac{P_2}{A_2} = \frac{a_2 b_2}{A_2}, \text{ etc.}$$

It still remains to determine A_1, A_2.

Now $$V = A\,l = A_1\,l_1 = A_2\,l_2$$

and $$l_1 = l + e_1 = l + \overline{a_0\,a_1}$$

$$l_2 = l + e_2 = l + \overline{a_0\,a_2}$$

Then $$A_1 = \frac{V}{l + a_0\,a_1}$$

$$A_2 = \frac{V}{l + a_0\,a_2}$$

Consequently—

$$\Phi_1 N = \frac{a_1 b_1}{V}(l + a_0 a_1) \text{ and this may } = \overline{a_1 m_1}$$

$$\Phi_2 N = \frac{a_2 b_2}{V}(l + a_0 a_2) \text{ and this may } = \overline{a_2 m_2}$$

If we plot these values of ΦN as forces $\overline{a_1 m_1}$, $\overline{a_2 m_2}$, . . . after the respective divisions perpendicularly and longitudinally and rejoin the extremities m, m_1, m_2, m_3, . . . by a curve, we shall obtain the diagram of internal friction (cf. line ΦN in Fig. 44) plotted in relation to the elongation of the test-piece ($e = \psi l$). If on the contrary we wish to represent the friction only in function with the elongation in percentages (ψ) we shall have a diagram in which the absolute individual properties of the material are related to one another.

For this result it becomes necessary, starting from the values of the tension in percentages, *i.e.* of the ψ_1, ψ_2, ψ_3, . . . respectively, and from the already known values of $\Phi_1 N$, $\Phi_2 N$, $\Phi_3 N$, . . . to construct a new curve. The values of ψ_1, ψ_2, ψ_3, . . . are calculated in percentages by the formula

$$\psi \% = 100 \frac{\overline{a_0 a_1}}{l}$$

Then in Fig. 45 let $\overline{a_0 a_1} = \psi_1$; $\overline{a_0 a_2} = \psi_2$; $a_0 a_6 = \psi_z$.

If we drop perpendiculars from the points a_0, a_1, a_2 (Fig. 45), and if we project on to the latter the respective values of the internal friction of Fig. 44, *i.e.* the values $\overline{a_1 m_1}$, $\overline{a_2 m_2}$, . . ., we obtain in Fig. 45 the points m_1, m_2, m_3 . . .; we rejoin them into a curve. The curve ΦN thus obtained (Fig. 45) shows the law of the variation of the internal friction related to the elongation in percentages. The diagram represented by this curve shows us the absolute properties of the material which enter into consideration at each effort; it is for this reason that with this diagram we can calculate all sorts of forces, *i.e.* represent graphically the diagrams of all the forces to which materials are subjected.

Let us now examine two kinds of forces.

(*a*) *Tension.*—Our starting formula was—

$$p_{n_1} = \Phi_1 N = \frac{P_1}{A_1}, \text{ whence } P_1 = A_1 \Phi_1 N$$

and since, until the maximum force of tension is reached, we may consider the elongation as proportional and that the volume remains constant, we have

$$V = A_0 l_0 = A_1 l_1 = A_1 l_0 (1 + \psi_1)$$

and

$$A_1 = \frac{A_0}{1 + \psi_1}$$

Consequently—

$$P_1 = \frac{A_0}{1 + \psi_1} \Phi N \text{ and } P_{max.} = \frac{A_0}{1 + \psi_1} \Phi_{max.} N$$

If using the curve of the internal friction we measure the values of the internal friction $\Phi_1 N, \Phi_2 N, \ldots$ corresponding to $\psi_1, \psi_2, \ldots$ we shall be able to calculate the values $P_1, P_2, \ldots$ and consequently with the values of the elongation of the bar we can construct the tension diagram.

Instead of using this graphic method, we can also use an analytic method, the curves of the internal friction of the tenacious materials corresponding to a parabola whose vertex is at the extremity of the initial hardness $\Phi_0 N$.

If we grant that the origin of the co-ordinates becomes identical with the first point of the curve the formula for the parabola will be—

$$(y - Y_0)^2 = 2 p x$$

and $2p$ = the double of the parameter, which consequently we have to determine by experiment.

The formula for the internal friction is therefore—

$$\Phi_1 N = \Phi_0 N + \sqrt{2 p \psi_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

It is well to observe that the degree of tenacity may be represented by the part of the diagram of internal friction which is limited by $\Phi_{max.} N$. To obtain the value of this surface we integrate its differential between two determined limits. It results then—

$$a = \Phi_0 N \psi_z + \tfrac{2}{3} \sqrt{2 p \psi_z^{\frac{3}{2}}}$$

or

$$a^{\text{mkg./cm.}^3} = \Phi_0 N \psi_z + 0{\cdot}666 \sqrt{2 p \psi_z^3} \quad . \quad . \quad (2)$$

This is the formula of tenacity.

If we wish to obtain the value of the Formula (2) in mkg., we should admit that the diagram represents the tenacity of a bar whose section is of 1 mm.² and length 1 mm.; and its volume consequently 1 cm.².

If we wish to calculate Hartig's modulus of work of the tenacity, we should first express the tenacity of a bar of 1 dm.³ in volume, *i.e.* multiply the number obtained from Equation (2) by 1,000 and then divide by the specific gravity γ. We have then—

$$H^{\text{mkg. cm.}^3} = 1000\, a \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

and

$$H^{\text{mkg./cm.}^3} = \frac{1000\, a^{\text{mkg. cm.}^2}}{\gamma} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The analytic formula of the force of tension is obtained by introducing into the formula hitherto known for the tensile stress for long bars of tenacious material—

$$p_1 = A_0 \frac{\Phi_1 N}{1 + \psi_1}$$

instead of Φ N the value given by Equation (1).

We have then for the tensile stress the following equation :—

$$p_1 = A \frac{\Phi_0 N + \sqrt{2 p \psi_1}}{1 + \psi_1} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

and for the maximum tensile stress—

$$p_{\max.} = A \frac{\Phi_0 N + \sqrt{2 p \psi_z}}{1 + \psi_z} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5a)$$

The Equation (5) is none other than the equation of the tension diagram.

In order to obtain the value of the surfaces of these diagrams, *i.e.* the work effected by the tensile force, we must find its differential, which we obtain by multiplying the force of tension corresponding to $\psi_1 = x$ by the differential of the road surveyed and then integrate between the limits $x = 0$ and $x = \psi_z$. It follows then—

$$a = A\, l \left[\Phi_0 N \int_0^{\psi_z} \frac{dx}{1+x} + \sqrt{2p} \int_0^{\psi_z} \frac{\sqrt{x}}{1+x} dx \right]$$

In substituting the values of the integral we shall have as the force of tension developed up to its limit the following formula :—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} h^{\text{m.}} \Big[\Phi_0 N \,.\, 2{\cdot}3 \log (1 + \psi_z)$$
$$+ 2\sqrt{2p} \left(\sqrt{\psi_z} - \text{arc. tan} \sqrt{\psi_z} \right) \Big] \quad . \quad . \quad (6)$$

With what has already been said we cannot yet determine the formulæ of the other kinds of forces. We have yet to introduce what is called the angle of molecular action, which is also a characteristic property of the substance. We know that at the time of the deformation not only the particles of the body, which are directly submitted to the influence of the external force, slide, but also many others, because the particles of the body are in a condition to transmit the force. The transmission of the influence of the external force is made, as we know, by means of the internal forces, and consequently it is necessary that the direction of the transmission of the force should coincide with the direction of the internal forces. Since the internal forces depend on the masses, we must admit that they have the centre of gravity of the molecules as the point of application and, as their direction, the line of junction of the centres of gravity of the molecules.

The lines of junction of the centres of gravity of the molecules form a regular trellis, whose section with a plane represents a network, as is

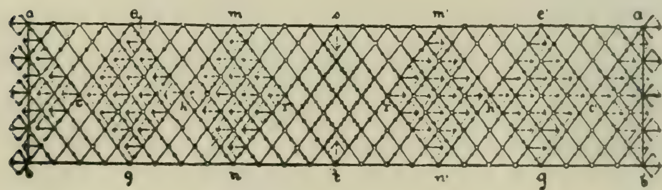


FIG. 46.

shown in Fig. 46. We see that in the figure showing the network of the forces, the transmission of force in the plane follows the lines of junction of the centres of gravity. Since the direction of these lines of junction depends on the form of the molecules, and since this latter is characteristic for the same substance, we may say that the direction of the line of junction, and consequently also the direction of the transmission of the force, is characteristic for the same substance.

We will call the angle of the direction of the transmission of the force with the direction in which the augmentation of the dimensions takes place, *the angle of the molecular action*, and we will always designate it by β in future.

We can with the help of the angle β determine all the deformations and for the different forces the relation between the external force and the extending action. Before passing on to the other forces we will show briefly the transmission of the force in tensile tests.

Suppose the test-piece has a cylindrical shape, and that the tensile force is uniformly distributed, and further that the tensile force only increases slowly, so that in very short intervals of time it may be con-

sidered as constant. While the propagation of the external force is situated in the direction of the angle of molecular action, let us first, before everything else, draw the molecular network of our pattern, as it is shown in Fig. 46, which represents the bar in longitudinal section.

The equally distributed tensile stresses act upon the two bases $a\ b$ and $a'\ b'$. As the molecules can only transmit the force in the direction of the angle of molecular action, it is necessary first to decompose the external forces into components whose direction coincides with that of the angle of molecular action. Each component acts upon each of the molecules in its direction, which we can represent by lines of force. Consequently let us draw the lines of force of the extreme points in the direction of the angle of molecular action, *i.e.* starting from the point a and going as far as g, from the point b as far as e, then from a' to g' and from b' to e', then from the remaining molecules of the ends in each direction of the angle of molecular action, as is shown in Fig. 46.

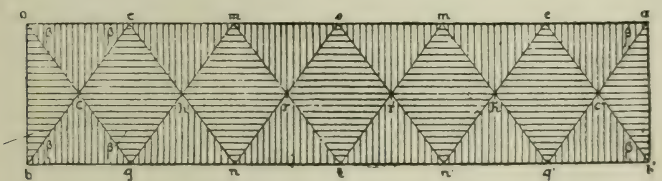


FIG. 47.

Considering those lines of force more closely, we shall see that upon each molecule in the regions $a\ b\ c$ and $a'\ b'\ c'$ are exercised two forces whose resultant has the same direction as that of the tensile force; that is why these particles may be considered as united to the external force.

Further, we observe that upon the molecules in the regions $a\ c\ e$, $b\ c\ g$, $a'\ c'\ g'$ and $b'\ c'\ g'$, only one component is exercised, and that in the direction of the angle of molecular action, so that these molecules have a tendency to slide in the direction of the increase and in that of decrease of the dimensions.

To this tendency the internal friction is opposed. So long as the latter is greater than the action of the external force, the molecules will not change position; however, the friction $\Phi\,N$ of these particles will be greater than that of the neighbouring molecules upon which the action of the force has not yet been transmitted, because the action of the external force increases the normal pressure N between the various particles.

By virtue of this greater internal friction, these particles will be more intimately united than those which are placed between them, and it is for that reason that we may consider them as united to the external force.

In the succeeding period of propagation of the action of the force of tension, we should consider the molecules in the surfaces $e\,c\,g$ and $e'\,c'\,g'$ as points of application of the external forces, and for this motive we ought, starting from these molecules, to carry the lines of force exactly as for the molecules of the ends. We continue thus, until finally the primative tensile force acts upon all the molecules of the test-piece.

We see that in Fig. 46 the parts of the test-piece submitted to the force are limited by the sides of the angle of molecular action, drawn from the extreme points. Consequently we have only to draw straight lines from the extreme points $(a\,b,\,a'\,b')$ under the angle β, and that as far as the point of intersection of the superior or inferior limit of the bar in e' and g'. From each of the points of intersection we again draw a straight line under the same angle as it is represented in Fig. 47. We pursue this construction in starting from the two ends until we arrive at the middle of the length. In order to demonstrate the force to which the particles which take the direction of the external force are submitted, we will provide them (see Fig. 47) with horizontal etchings, and those

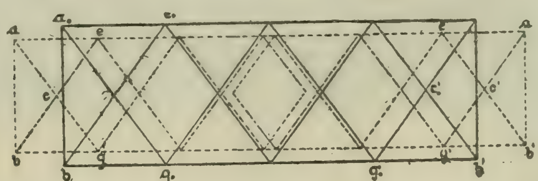


FIG. 48.

which the forces tend to deform we will represent by vertical etchings. We see then that so long as the external force in its action is inferior to that of the internal friction, this action extends over the whole bar without causing a permanent displacement of the molecules.

So soon as the action of the external force is greater than that of the internal friction, and if the material is tenacious, the difference between the two actions will have the effect of moving the parts $a\;b\;c$ and $a'\;b'\;c'$ (Fig. 47) in the direction of the tensile force as if the particles were united to it, while the parts $a\;c\;e$, $b\;c\;g$, $a'\;c'\;e'$, and $b'\;c'\;g'$ will be displaced because they are submitted to components inclined against the force.

By reason of the displacement there will follow according to our hypothesis for these parts an elevation of the coefficient of friction (Φ) and consequently the action of the friction will again be greater than that of the external force. The particles thus considered will be more intimately united by this great friction than the neighbouring particles and consequently will propagate the action of the external force. The parts $c\;g\;h\;e$ and $c'\;g'\;h'\;e'$ will accordingly take the direction of the force

of tension ; contrarily the parts $e\,h\,m$, $g\,h\,n$ and $e'\,h'\,m'$, $g'\,h'\,n'$, will be deformed until they are more intimately united by the increasing action of the friction and thus rendered capable of transmitting the action of the external force.

The parts represented by the vertical etchings transmit thus continually the action of the external force, until the whole bar is submitted to this action ; it is only then that the external force is able to increase.

If we again grant that this excess of force remains constant during a very short time, its action will again be transmitted in a known way to the entire bar. Under the influence of this excess of force there will be some particles which will again be united, but others will be displaced, and these are not identical with the ones which were united or displaced before by the primative force, because through the first deformation the bar became longer and thinner ; consequently the new network of the lines of force cannot coincide with the preceding one, as is shown in Fig. 48.

It follows from the new position of the lines of force that from the deformation which will take place, certain molecules can be united, which have formerly been displaced, and others which have before been united may now be displaced.

If we imagine the force going thus while increasing, a moment will come when, according to the continual displacement of the lines of force, almost all the molecules will have been displaced, and consequently the bar will be almost uniformly deformed over its whole length. From this it follows that :—

The uniform deformation only exists so long as the coefficient of friction increases and the internal friction is not greater than the cohesion.

It is only in the middle of the bar that there can be irregularities of lengthening, and these are also found in very short bars. In order to diminish the influence of these irregularities we can make these tensile tests on long bars.

For plastic substances, where the coefficient of friction is constant, the transmission of force cannot be made in the way indicated, because so soon as one of the particles represented by vertical etchings, of length $\delta \cot \beta$ is displaced, the force exerted on this particle increases ; consequently it will undergo a fresh deformation without transmitting the force to the other particles ; in other words, the bar will undergo a striction.

In order to understand the influence of other kinds of forces, it is not necessary to know the other consequences of the tensile force ; therefore we will pass on to the compressive forces.

(*b*) *Compression.*—In order to elucidate the force of compression grant : (1) that the compressive forces act, uniformly distributed, upon a cylinder with a circular base ; (2) that this external force only increases slowly, and consequently is constant in a very short interval, and exerts no shock ; (3) that the material of the cylinder is incompressible.

As the transmission of the external force only exists in the direction of the angle of action we ought at first to establish the molecular network,

where we must take into consideration that the angle of action β is formed by the direction of the force and the direction of the increase of dimensions.

In the case of compressive forces the decrease of dimensions is situated in the direction of the external force, and the increase of dimensions in a direction normal to the external force. The angle of action β is therefore the angle between the direction of the molecular action and the normal to the force (Fig. 49).

Now, as the molecules only transmit the action of the external force in

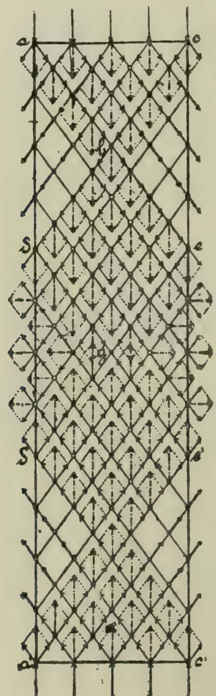


FIG. 49.

the direction of the angle of action, we shall have to determine the components of the external forces, acting in the direction of the angle of action.

In order to represent this force by a diagram we shall have to designate the action of each component upon the molecules by lines of force acting each in their proper direction as was done for the tensile forces.

Consequently, we will carry (Fig. 49) the straight lines which repre-

sent the force of a as far as e, of c to d, of a' to e', of c' to d' and further of the molecules found between a and c and a' and c' in the direction of the angle of action, respectively. It follows from the figure that upon the molecules $a\ b\ c$ and $a'\ b'\ c'$ two forces are exerted whose resultant falls on the direction of the external force ; it is for this reason that we may consider them as united, fixed to the point of application of the external force. Contrarily upon the molecules the parts $a\ b\ d$, $c\ b\ e$ and $a'\ b'\ d'$, $c'\ b'\ e'$ the forces only act in one direction. These molecules have a tendency to move in the direction of the increase of dimensions ; they are therefore subjected to the extending action resulting from the compressive force.

$\varepsilon > 2\ D \tan \beta.$

FIG. 50.

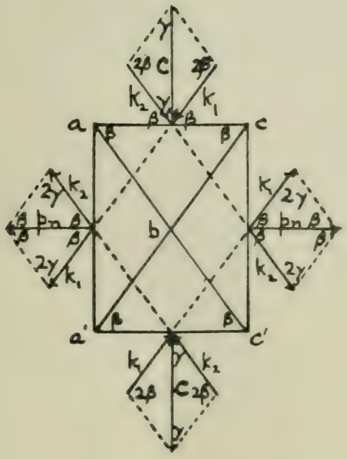


FIG. 51.

So long as the internal friction is greater than the extending action resulting from the compressive forces the molecules of the parts $a\ b\ d$, $c\ b\ e$, $a'\ b'\ d'$ and $c'\ b'\ e'$ will remain motionless ; but as by virtue of the increase of normal pressure N the internal friction of these parts increases, they are more strongly retained than those on which the external force is not yet acting, and it is for that reason that, as for the force of tension, we ought to consider them as the parts complementary to the active force.

In the succeeding period of the forces, we must consider the molecules of the bases $d\ b\ e$ and $d'\ b'\ e'$ as points of application of the external force, and for this reason we ought to carry the action of the external force from these points as formerly for the points of the surface $a\ c$.

We continue thus, as for the force of tension. We see then in the figure thus completed that the parts differently submitted to the force are bounded by the sides of the angle drawn from the extreme points, and we can thus continue as for tension ; only the parts which are retained will be represented by vertical etchings, and the others by horizontal etchings (Fig. 50).

This figure shows that the parts (Fig. 50) represented by horizontal etchings are knitted between the particles united in the same way, thus being maintained in equilibrium ; they have therefore the same position as in Fig. 51. For this reason we can determine the extending action as follows :—

We can divide the pressure c and the force of reaction, each one following the direction of the propagation of the force, into two components, k_1 and k_2. These components act on the lateral forces and are the cause of the extension (swelling) p_n, whose value is deduced from Fig 51.

$$p_n = k_1 \cos\beta + k_2 \cos\beta$$

and as

$$k_1 = k_2$$

it follows that

$$p_n = 2\, k_1 \cos\beta$$

But according to the same figure—

$$c_n = k_1 \cos\gamma + k_2 \cos\gamma$$

$$= 2\, k_1 \cos\gamma$$

then

$$k_1 = \frac{c}{2\cos\gamma}$$

Now as

$$2\gamma + 2\beta = 180^\circ$$

or

$$\gamma + \beta = 90^\circ$$

and

$$\cos\gamma = \sin\beta$$

we have

$$k_1 = \frac{c}{2\sin\beta}$$

We have then

$$p_n = \frac{2\cos\beta \,.\, c}{2\sin\beta} = c\cot\beta$$

According to our preliminary formula, the extending action is in equilibrium with the internal friction.

$$p_n = \Phi\mathrm{N}, \quad \text{then} \quad \Phi\mathrm{N} = c\cot\beta$$

or

$$c = \frac{\Phi\mathrm{N}}{\cot\beta} = \Phi\mathrm{N}\tan\beta$$

Knowing the relation of the internal friction to the force of compression, we will examine the deformation of *plastic materials.*

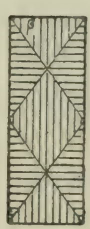
As the extending action of the external force ($c \cot \beta$) is smaller than that of the internal friction, so the action of the external force is propagated over the entire bar without causing permanent displacement. But if the action of the friction is exceeded by that of the external force, *i.e.* if—

$$c \cot \beta > \Phi \, \mathrm{N}$$

the excess of force will only have action for the moment as far as the points $d\, e$ and $d'\, e'$ (Fig. 49) commencing with the molecules of the bases $a\, c$ and $a'\, c'$. Consequently the particles $a\, b\, c$ and $a'\, b'\, c'$ will follow the direction of the force; the parts $a\, b\, d$, $c\, b\, e$ and $a'\, b'\, d'$, and $c'\, b'\, e'$ on the contrary will be displaced. This displacement may take place without rupture, because according to our hypothesis the cohesion is much greater than the internal friction while the extending action is only a little greater than the internal friction.

$\delta = \mathrm{D} \tan \beta.$

FIG. 52.



$\delta = 2\, \mathrm{D} \tan \beta.$

FIG. 53.

From what we have said up till now it follows that the effect of the surplus of the force only extends over the parts $a\, d\, b\, e\, c$ and $a'\, d'\, b'\, e'\, c'$, and since the parts $a\, b\, d$, $c\, b\, e$ and $a'\, b'\, d'$, $c'\, b'\, e'$ utilize the excess of force for the acceleration of their molecules, the action of the external forces cannot be transmitted to the other parts of the body so long as these latter have undergone no considerable amplification of their section.

We may say then that, when we compress long cylinders, the particles in the direct neighbourhood of the surfaces of pressure are deformed a length $a\, d = a'\, d' = \mathrm{D} \,.\, \tan \beta$ more than the others, *i.e.* the cylinder will be constrained at its two ends.

It is not only relatively high cylinders which present the deformation of the ends (*e.g.* in Fig. 50), but this phenomenon appears every time the

extreme parts are free on a length D tan β, therefore everywhere where—

$$\delta \geqq 2 \mathrm{D} \tan \beta$$

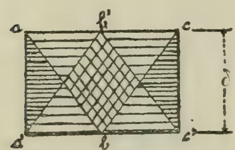
if δ designates the height of the piece.

But as soon as the height of the cylinder is smaller than 2 D tan β the constrained end parts are penetrated, and the bar is deformed over the whole length and that as much more uniformly as the penetration is more complete.

The penetration is complete as soon as—

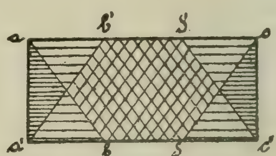
$$\delta = \mathrm{D} \tan \beta$$

as it appears in Fig. 52. In that case there will be a very pronounced deformation. Fig. 54 shows that if the height of the bar δ is $\leqq$ 0·5 D tan β a friction will be produced between the instrument used and the body



$\delta \leqq 0{\cdot}5\ \mathrm{D} \tan \beta.$

FIG. 54.



$\delta > 0{\cdot}5\ \mathrm{D} \tan \beta.$

FIG. 55.

submitted to the force; further, Fig. 55 shows that in the case where $\delta > 0{\cdot}5$ D tan β there will be formed, between the surfaces where the pressure is exerted, united molecular groups, which for their displacement require besides the action of the force of pressure uniformly distributed, another force of pressure. In this case the resistance to the compression of the material is greater and less uniformly distributed over the surface where it is exerted.

Knowing this general rule, we can now calculate the force of compression. In every case where—

$$\delta > 0{\cdot}5\ \mathrm{D} \tan \beta$$

we have as the equation of equilibrium—

$$c \cot \beta = \Phi \mathrm{N}$$

and consequently the pressure per unit of section, *i.e.* the resistance to the compression, will have the value—

$$c = \frac{\Phi \mathrm{N}}{\cot \beta} = \Phi \mathrm{N} \tan \beta$$

and if we designate by A_x the actual surface of pressure, we shall have as the absolute force of pressure—

$$C_x = A_x . \Phi N \tan \beta = A_x c.$$

This formula, then, does not allow us to calculate the exact value of the pressure except when we know exactly the variation of A_x.

In the cases where $\delta < 2 D \tan \beta$

and particularly when $\delta < 1{\cdot}5 D \tan \beta$

we may, without committing any great error, consider the deformation as sufficiently uniform. In this case we shall be able to calculate the variation of the section, since the volume (V) is constant.

Now $V = A_0 \delta_0 = A_1 \delta_1 = A_2 \delta_2 = \ldots A_x \delta_x$, etc.,

where A_0 and δ_0 designate the initial dimensions, and the other values of A and δ the deformed dimensions.

We will now again study the deformations of the tenacious materials.

So long as the extending action of the external force ($c \cot \beta$) is smaller than that of the internal friction, the former is propagated over the entire piece, without causing deformation. But so soon as the action of swelling is greater than the internal friction, the molecules of the parts $a\ b\ c$ and $a'\ b'\ c'$ of Fig. 49 are retained, the molecules of the parts $a\ b\ d$, $c\ b\ e$, $a'\ b'\ d'$, $c'\ b'\ e'$ on the contrary displaced. According to our hypothesis, by virtue of the displacement of the molecules the coefficient of friction will increase, and in these molecules the friction will consequently become greater with the extending action of the external force, and accordingly the molecules are retained and rendered capable of transmitting the external force. When the external force has been transmitted by these molecules to the other parts of the piece the action of the external force has the effect of retaining the particles $d\ b\ e\ g$ and $d'\ b'\ e'\ g'$, and displaces the particles $d\ g\ d'$ and $e\ g\ e'$ until they are retained by the thus increasing friction. If the bar is still longer, these united particles will continually transmit the action of the external force until it is spread out over the entire bar.

So soon as the action of the initial external force is spread out over the entire bar we can let the force increase, and, if it remains constant during a very short time, its action will be transmitted in the same way to the piece.

So long as the coefficient of friction increases, the action of the external force, and consequently also the deformation, will be propagated uniformly to the whole bar (except in the case of the folding up of long bars), and for this reason we will call the latter the uniform deformation corresponding to the tenacity.

For the uniform deformation we can always calculate the surface of pressure of the volume which remains constant, because—

$$V = A_0 \delta_0 = A_1 \delta_1 = A_2 \delta_2 = A_x \delta_x$$

and since $$D_1 = D_0 (1 + \psi)$$

it follows that $$A_1 = A_0 (1 + \psi)^2.$$

We ought then to consider the surface of pressure as known, and if we still take account of the fact that the extending action of the external force ought to be equal to the internal friction modified, we shall have—

$$c_0 \cot \beta = \Phi_0 N$$

$$c_1 \cot \beta = \Phi_1 N, \text{ etc.}$$

consequently
$$C_1 = A_1 \Phi_1 N \tan \beta$$

$$C_1 = A_0 (1 + \psi)^2 \Phi_1 N \tan \beta$$

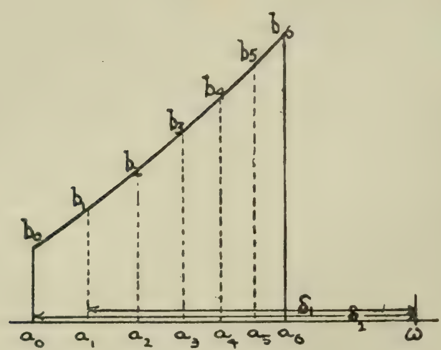


FIG. 56.

Using the diagram of internal friction, which we have deduced from the force of tension, we shall be able, so soon as we know the angle of action β, to calculate the force of pressure by this formula.

Now, we determine this value by means of an ordinate and an abscissa of a diagram of pressure for the case where $\delta > 0{\cdot}5$ D $\tan \beta$ because

then— $$C_1 = A_1 \Phi_1 N \tan \beta$$

therefore $$\tan \beta = \frac{C_1}{A_1} \Phi_1 N$$

(See Fig. 56, where $\overline{a_0 a_1} = \delta_0 - \delta_1$ and $\overline{b_1 a_1} = C_1$.)

In applying the formula formerly found for the compression we shall need the curve of internal friction.

If, on the contrary, we want to use the analytic method, we shall have to substitute in the formula obtained for compression the analytic value of Φ_{I} N. We shall then have—

$$C_{I} = A_{0}(1+\psi_{I})^{2}(\Phi_{0} N + \sqrt{2 p \psi_{I}}) \tan \beta \qquad (7)$$

This formula contains a variable ψ, represented in the curve by $\psi = \delta_{0} - \delta_{I}$. Introducing this value into the preceding formula and effecting some transformations on it we obtain as the formula of the pressure diagram—

$$C_{I} = A_{0} \frac{\delta_{0}}{\delta_{I}}\left[\Phi_{0} N + \sqrt{2 p \left(\sqrt{\frac{\delta_{0}}{\delta_{I}}} - 1\right)}\right] \tan \beta \qquad (8)$$

In order to obtain the work performed by the compression, we ought to multiply the value of C_{I} by the differential of the road surveyed and then integrate between the limits $x = \delta$ as far as $x = \delta_{I}$. Now we have—

$$a = A\, \delta \tan \beta \left[\Phi_{0} N \int_{\delta_{I}}^{\delta} \frac{d x}{x} + \sqrt{2 p} \int_{\delta_{I}}^{\delta} \sqrt{\sqrt{\frac{\delta}{\delta_{I}}} - 1} \cdot \frac{d x}{x}\right] \qquad (9)$$

Substituting the values of the two definite integrals we shall have as $\delta_{I} \gtreqless \delta_{z}$ for the work effected by the compression the following expression—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} \delta^{\text{m.}} \tan \beta \left[\Phi_{0} N\, 2{\cdot}3 \log \frac{\delta}{\delta_{I}}\right.$$
$$\left. + 4 \sqrt{2 p}\left(\sqrt{\sqrt{\frac{\delta}{\delta_{I}}} - 1} - \text{arc tan} \sqrt{\sqrt{\frac{\delta}{\delta_{I}}} - 1}\right)\right] \qquad (10)$$

As in many cases the application of the formula becomes easier when in the place of $\frac{\delta}{\delta_{I}}$ we substitute the corresponding values of ψ, we can, making use of the Formula (9) bring the Equation (10) into the following form—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} \delta^{\text{m.}} \tan \beta \left[\Phi_{0} N\, 2{\cdot}3 \log \left(1 + \psi_{I}\right)^{2}\right.$$
$$\left. + 4 \sqrt{2 p}\left(\sqrt{\psi_{I}} - \text{arc tan} \sqrt{\psi_{I}}\right)\right] \quad . \qquad (11)$$

where $\psi_1 \leqq \psi_z$. Consequently we can express the work of compression necessary in order to reach the limit of elasticity as follows :—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} \delta^{\text{m.}} \tan\beta \Big[\Phi_0 N \, 2{\cdot}3 \log (1+\psi_z)^2 + 4\sqrt{2p}\left(\sqrt{\psi_z} - \text{arc tan} \sqrt{\psi_z}\right)\Big] \quad . \quad (12)$$

If the material is not only elastic, but also plastic, we shall have as the formula for the total work—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} \delta^{\text{m.}} \tan\beta \Big[\Phi_0 N \, 2{\cdot}3 \log (1+\psi_z)^2 + 4\sqrt{2p}\left(\sqrt{\psi_z} - \text{arc tan} \sqrt{\psi_z}\right) + \left(\Phi N + \sqrt{2p\psi_z}\right) 2{\cdot}3 \log\left(\frac{1+\psi_1}{1+\psi_z}\right)^2 \Big] \quad (13)$$

But if the material is only plastic we shall have—

$$a^{\text{mkg.}} = A^{\text{mm.}^2} \delta^{\text{m.}} \tan\beta \; \Phi_0 N \, 2{\cdot}3 \log \frac{\delta}{\delta_1} \quad . \quad . \quad (14)$$

It follows from the above that it is possible to calculate the force and the work necessary for the primary forces, *i.e.* for tension and compression, simply with the help of the internal friction and angle of action.

APPENDIX VII.

On Friction with Cohesion.

It has been found from experiment that the friction which two solid bodies, with their surfaces in a given condition, are capable of exerting, is simply proportional to the force with which they are pressed together.

Hence it follows that the friction may be computed by multiplying the force with which the two bodies are pressed together by a constant coefficient which is to be determined by experiment, and which depends on the nature of the bodies and the condition of their surfaces. Thus if P denote the pressure, Φ the friction modulus, and F the force of friction, then $F = \Phi P$. Let A A in Fig. 57 be the surface of contact. $\overline{PB}$ represents in amount, direction, and position the resultant of a force P, by which the upper body is pushed obliquely towards the plane. B is the centre of pressure of the surface of contact.

Now resolve $\overline{PB}$ into two rectangular components: one $\overline{NB}$ normal to the plane of contact and pressing the bodies together, the other $\overline{TB}$

tangential to the plane of contact and tending to make the bodies slide on each other. Calling the normal component N and the tangential component T, we have for an angle of obliquity B P T or P B N = θ—

$$\left.\begin{aligned} N &= P \,.\, \cos\theta \\ T &= P \,.\, \sin\theta = N \,.\, \tan\theta \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

So long as the tangential force T is not greater than Φ N it will be balanced by the friction, which will be equal and opposite; but the

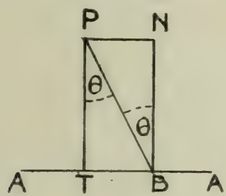


FIG. 57.

friction cannot exceed Φ N, so that if T exceed this limit it will be no longer balanced by the friction, but will cause the bodies to slide on each other. Evidently the condition that T shall not exceed Φ N is equivalent to the condition that $\frac{T}{N}$ or tan θ shall not exceed Φ, from which it follows that the greatest angle of obliquity of pressure between two planes which is consistent with stability is the angle whose tangent is the friction modulus. This angle is called the angle of repose, and is denoted by ϕ. It is the steepest inclination of a plane to the horizon at which the upper body will remain in equilibrium upon the plane; for if P represents the weight of the body, so that P B is vertical, and $\theta = \phi$, then ϕ is the inclination of A A to the horizon.

The following equations define the relations between the friction, the normal pressure, and the total pressure when the obliquity is equal to the angle of repose:—

$$F = T = \Phi N = N \,.\, \tan\phi = P \,.\, \sin\phi = \frac{\Phi P}{\sqrt{1 + \Phi^2}} \quad . \quad (2)$$

In a paper on "Reinforced Concrete Bins," recorded in the *Transactions of the Concrete Institute*, Vol. II, Part II, pages 139–218, the author gave values of the friction modulus for various materials but not for steel in contact with concrete. For lack of experiments on

concrete, it may be recorded here that the coefficient of friction for wrought iron on

Soft limestone, well dressed, is	0·49
Hard limestone, well dressed, is	0·24
Hard limestone, well dressed, wet, is	0·30

The friction modulus for concrete blocks on concrete blocks is about 0·65.

In the paper above referred to the theory of the frictional stability of a granular mass devoid of cohesion was dealt with at length, including Rankine's analysis, which is based upon the resistance to sliding along a given plane in a loose, granular mass being equal to the normal pressure exerted between the parts of the mass on either side of that plane, multiplied by a specified constant, which is the friction modulus of the mass and the tangent of the angle of repose.

If p_n denotes the normal intensity of pressure on the plane of rupture, and f the intensity of resistance to sliding, the symbolical expression of this principle is $f/p_n = \tan \phi$.

It follows from the above-stated principle, and from the principle that in a granular mass any plane may be considered as a plane joint, that the condition of stability will require that the direction of pressure, between the portions into which the mass is divided by the plane, shall not, at any point, make with the normal to that plane an angle exceeding the angle of repose.

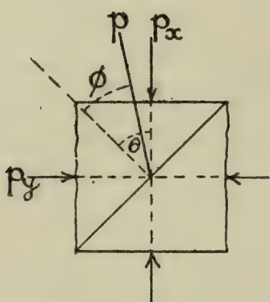


FIG. 58.

It has already been demonstrated (Appendix V, Equation 22) that if the intensities of two principal stresses at a point are p_x and p_y then—

$$\frac{p_y}{p_x} = \frac{1 - \sin \phi}{1 + \sin \phi}$$

where ϕ is the angle which the resultant stress, on any intermediate plane whose normal is inclined at the angle θ to the axis of p_x, makes with the normal (see Fig. 58).

In the case of a granular mass a limit is set to the angle of the resultant stress, namely that it shall not exceed the angle of repose.

Therefore in a granular mass

$$\frac{p_y}{p_x} \leqq \frac{1-\sin\phi}{1+\sin\phi}$$

where the symbol $\leqq$ signifies "less than or equal to," *i.e.* "not greater than."

Seeing that the stress p_x stands to the stress p_y as cause to effect, the first is frequently called the active stress and the second passive stress. As the passive forces are caused by the application of the active forces to the body or structure evidently they will not increase after the active forces have been balanced by them, and will therefore not increase beyond the least amount capable of balancing the active forces, so that we can write—

$$p_y = p_x \frac{1-\sin\phi}{1+\sin\phi}$$

The effect of cohesion in a granular mass will now be considered.

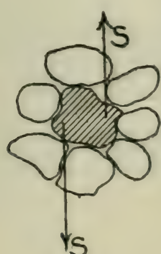


FIG. 59.

Let us first take the simplest case, namely a pure shearing stress s acting upon the very small grain shown magnified in Fig. 59. Now make the assumption that the granular particle is stronger than the cementitious medium in which it is embedded, and that the medium is elastic so as to give sufficiently for both the tensile resistance of the cement and the friction of the grain (against the other grains with which it is in contact) to occur at one and the same time.

Figs. 60, 61 and 62 show how such shearing forces acting on the medium that embeds regular and irregular grains result in producing compressive and tensile stresses normal to the surface of the grain at every part. The compressive stresses induce friction.

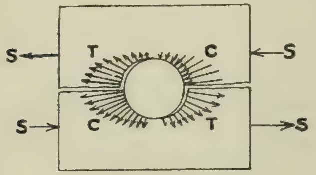


FIG. 60.

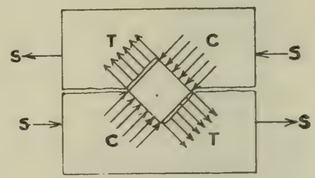


FIG. 61.

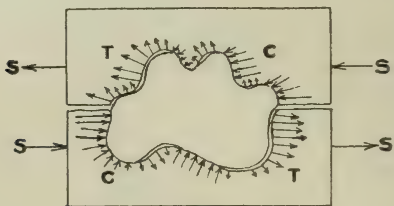


FIG. 62.

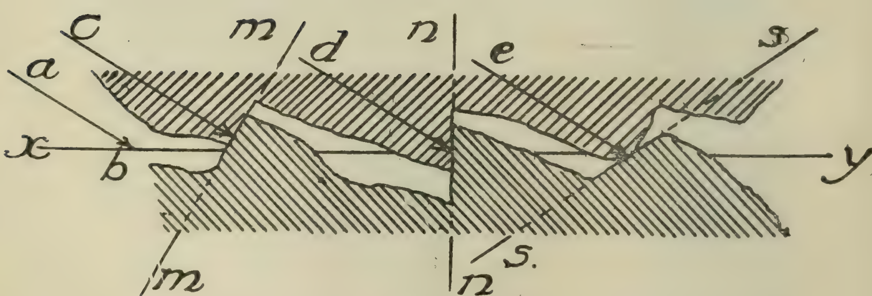


FIG. 63.

The following is quoted from Trautwine's "Civil Engineers' Pocket-Book" (Chapman and Hall, Ltd.) :—

"Even the most highly-polished flat surface, as $x\,y$, Fig. 63, is not (as it appears to the eye) a *plane*, but it is, in fact, a more or less jagged surface, as would appear under a sufficiently powerful microscope ; so that the force $a\,b$, instead of forming the *apparent* angle $a\,b\,x$, with *one* smooth surface, $x\,y$, of application, really becomes a series of parallel forces, as c, d, and e, which form other angles with a number of surfaces, $m\,m$, $n\,n$,

etc., of application, *inclined* (often in different directions) to the general surface, $x\,y$, as shown. Among these surfaces may be some, as $m\,m$, at right angles to the applied force ; and the force c will be imparted to them in its original direction, although applied *obliquely* to the *apparent* surface $x\,y$. In the case of the two forces d and e, applied to the surfaces $n\,n$ and $s\,s$, if the sliding tendencies along the two surfaces are *equal* and act in *opposition* to each other, the *combined* resistance of the two surfaces $n\,n$ and $s\,s$, is directly opposite to the forces, as would be that of a single surface at right angles to those forces. It is of course entirely out of the question to ascertain the exact resistance of each such microscopic projection in any given case. Instead of this, we find by experiment the *combined* resistance which *all* of the projections, in a given case, offer to the sliding force, and give to this resistance the name of *friction*."

The foregoing remarks will equally well apply to the tensile forces acting through a plane surface within a cohesive material. If we test such a material in direct tension it will sever at approximately right angles to the direction of the pull, and its tensile stress is computed by dividing the total force by the area of the sectional plane at right angles to the direction of the applied force. As our frictional and cohesive constants are thus average values derived from plane surfaces tangential and at right angles respectively to the forces which cause frictional and cohesive stresses, we may legitimately consider the average effect of shear on a grain by analysing the stresses on plane surfaces. First let the shearing force on the average grain be replaced by the tensile and compressive components at 45° (which, as explained in Appendix V, Formula (18a), are the equivalent of a shearing stress), as shown in Fig. 64. The face $a\,b$ of the wedge element surrounding the grain is then the average surface of friction and the face $b\,c$ is the average surface of cohesion.

Take the area of the face $a\,b$ as unity. Then referring to Fig. 65, let t and c be the tensile and compressive components of the shear stress, and r = the cohesive resistance of the material and Φ = the friction modulus.

Now, as explained above, $c = t = s$, $\therefore$ $T = s$, $C = s$, $R = r$, and $F = \Phi\,C = \Phi\,s$. We have for equilibrum $T = R + F$.

$$\therefore s = r + \Phi\,s$$

$$\therefore s\,(1 - \Phi) = r$$

and
$$s = \frac{r}{1 - \Phi}$$

If $\Phi = 0{\cdot}65$ then $s = 2{\cdot}85\,r$ and $r = 0{\cdot}35\,s$.

That is to say, the pure shear resistance is about three times the tensile resistance, which as will be seen later amounts for the same value of Φ to one-tenth the compressive resistance, thus giving the shear strength as about one-third the compressive strength.

It is interesting to compare with tests by Bouton and Bauschinger referred to by Professor Johnson. Bouton found that the angle of repose ϕ of limestone was 33·4° which coincides with a value of $\Phi = \tan\phi = 0·659$, while Bauschinger gives as the resistance of a uniform limestone :

Tensile strength, 213 lb./in.^{2}
Shearing strength, 583 lb./in.^{2}

By the above formula taking Φ as $= 0·659$ the shearing strength ought to work out at 625, assuming the tensile value is correct and the tension at 199 by the converse method. Considering the difficulty of

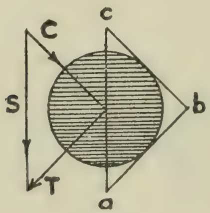


FIG. 64.

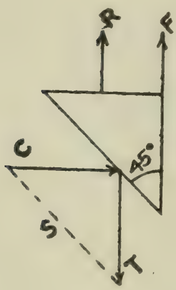


FIG. 65.

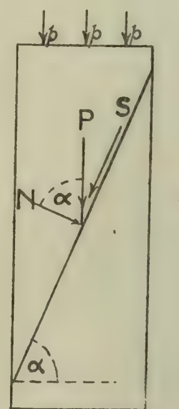


FIG. 66.

getting true values in tests for shear and tension this appears somewhere about right.

Now consider the combination of a normal and a tangential stress as regards rupture by shearing. The effect of the normal stress will be to increase the resistance to sliding upon the plane of rupture by causing additional friction thereon. Let N and S_1 be the normal and tangential forces, and let us treat them on similar lines to the foregoing. The active shearing force will evidently be reduced to—

$$S = S_1 - \Phi N$$

The face over which this residual shearing force acts is $\sqrt{2} \,.\, \overline{a\,b}$ which equals $\sqrt{2}$, where $\overline{a\,b}$ is unity. Consequently the stress—

$$s = \frac{S}{\sqrt{2}} = \frac{S_1 - \Phi N}{\sqrt{2}} = \frac{S_1 \sqrt{2} - \Phi n \sqrt{2}}{\sqrt{2}} = s_1 - \Phi n$$

As before $s = \frac{r}{1 - \Phi}$, so that—

$$s_1 - \Phi n = \frac{r}{1 - \Phi}$$

from which $$(s_1 - \Phi n)(1 - \Phi) = r \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and $$s_1 = \frac{r + \Phi(1 - \Phi) n}{1 - \Phi} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

We may now proceed a step further to the consideration of the compression of a fairly long prism of brittle material, which when it fails will develop an inclined plane of rupture. Referring to Fig. 66, we resolve the pressure P into its tangential and normal components on the plane of rupture, which is inclined at the angle α to the horizontal and to the direction of loading, thus getting—

$$N = P \cos \alpha \text{ and } S = P \sin \alpha$$

The area of the inclined plane $= A/\cos \alpha$, where A is the cross-sectional area of the prism.

By dividing the tangential and normal components by the area of the inclined plane and substituting for $P = p\,A$ we get the tangential component stress $s = p \sin \alpha \cos \alpha$ and the normal component stress $n = p \cos^2 \alpha$. Inserting these values in Equation (1) we have—

$$r = (1 - \Phi)\, p\, (\sin \alpha \cos \alpha - \Phi \cos^2 \alpha)$$

from which $$p = \frac{r}{(1 - \Phi)(\sin \alpha \cos \alpha - \Phi \cos^2 \alpha)} \quad . \quad . \quad . \quad . \quad (3)$$

It will be evident that the angle of rupture must be such as would cause failure under the least load, which by the foregoing formula is seen

to depend upon the value of α. The minimum value is determined by differentiating p in respect to α and equating to zero, thus—

$$\frac{dp}{d\alpha} = -\frac{r}{(1-\Phi)} \frac{(-\sin^2\alpha + \cos^2\alpha + 2\,\Phi\sin\alpha\cos\alpha)}{(\sin\alpha\cos\alpha - \Phi\cos^2\alpha)^2} = 0$$

from which $\sin^2\alpha - \cos^2\alpha = 2\,\Phi\sin\alpha\cos\alpha$

and $\Phi = \dfrac{\sin^2\alpha - \cos^2\alpha}{2\sin\alpha\cos\alpha} = -\dfrac{\cos 2\alpha}{\sin 2\alpha} = -\cot 2\alpha = \tan(2\alpha - 90^\circ)$

But $\Phi = \tan\phi$, therefore—

$$2\alpha - 90^\circ = \phi$$

or

$$\alpha = \frac{90^\circ + \phi}{2} = 45^\circ + \frac{\phi}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Taking the angle of repose for limestone as 33·4° as aforesaid we obtain $\alpha = 61{\cdot}7^\circ$, which agrees well with the experimental result.

Now insert the value of α in Equation (3), obtaining—

$$p = \frac{2r}{(1-\Phi)\,[\sin 2\alpha - \Phi(\cos 2\alpha + 1)]}$$

$$= \frac{2r}{(1-\Phi)\left(\dfrac{1}{\sqrt{\cot^2 2\alpha + 1}} - \dfrac{\Phi\cot 2\alpha}{\sqrt{\cot^2 2\alpha + 1}} - \Phi\right)}$$

$$= \frac{2r}{(1-\Phi)\left(\sqrt{1+\Phi^2} - \Phi\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

If $\Phi = 0{\cdot}659$ then $p = 10{\cdot}9\,r$.

As regards the shearing, compressive, and tensile resistances of cement mortar and concrete the density will greatly affect the result in the poorer mixtures. If the proportion of cement is ample rupture will occur by breakdown of the stone or other material of which the mortar or concrete is made, supplemented in varying degree by the network of neat cement surrounding the grains. The great size of the particles of coarse material in concrete will of course impart an element subject to considerable variation from the mean in our test specimens and in practice.

Professor Johnson in his book has worked out (by an approximate method) the maximum tensile stress in a cement briquette due to its

FIG. 67.

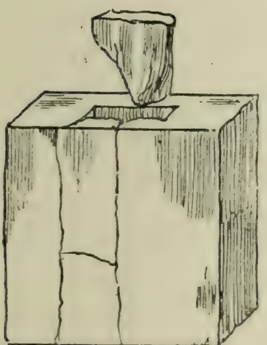


FIG. 68.

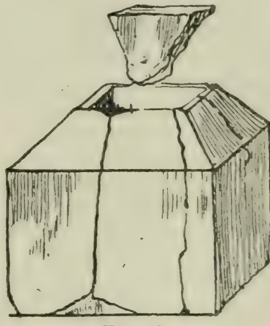


FIG. 69.

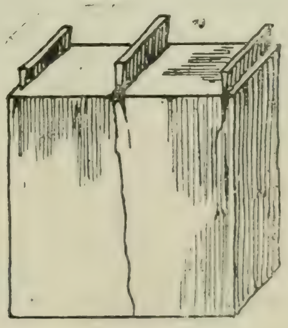


FIG. 70.

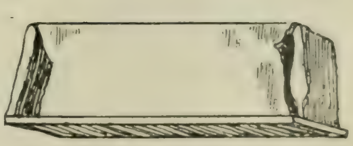


FIG. 71.

shape, and finds it to be 1·51 times the average stress, while Professor E. G. Coker has found by optical experiments that the maximum stress in the briquette form is 1·75 times the mean stress, but this relation is

only true within elastic limits.* The ascertained tensile strength from briquettes of mortar is generally about one-tenth to one-twelfth the compressive strength, and this after all is an average value, and constitutes the cohesive constant measured as explained on page 149.

The above results will only apply where the specimen is long enough to allow the ordinary fracture to be developed. When the specimen is reduced in height the strength is increased, the bearings of the compression machine gripping the ends by friction. Judging from tests it would seem that the tendency is to rupture on planes inclined at about the angle thus calculated.

Though the friction on the beds supplements the tensile resistance and increases the resistance to compression, as is shown by Figs. 65 to 69 of blocks of sandstone tested by Professor J. Bauschinger, redrawn from plates published in his classic *Communications from the Testing Laboratory of the School of Technology of Munich*, Fig. 67 shows the ordinary pyramid left by crushing a long prism whose faces have approximately the angle above calculated. Figs. 68, 69, and 70 show how when only portions of the surfaces are loaded the blocks are burst by lateral pressure developed by the sliding of an internal wedge on the plane of rupture. The specimen shown by Fig. 70 had the load applied at three places. The three wedges of stone that separated and split the block have for clearness been shown removed from the fractures and suspended above the test block. Fig. 71 is the view of a very short specimen, and here again the typical angle of rupture will be noticed at the edges of the specimen.

APPENDIX VIII.

Relation between Shearing Force and Bending Moment.

Let Fig. 72 represent the left-hand half of a beam loaded with any number of point loads of any intensity, namely, W_a, W_b, . . . to the left of portion A B C D and W_1, W_2, . . . on the portion itself. R_1 is the reaction at the support.

Now consider the equilibrium of the portion A B C D. At the section A B the shearing force will be—

$$S_1 = R_1 - \Sigma W_a$$

The mean value of the decrease of the shearing force on the length a will be—

$$\frac{\Sigma (W_1 x_1)}{a}$$

* See paper in *Proceedings of International Association for Testing Materials*, Vol. II. No. 10, June 20, 1912.

and the mean value of the total shearing force on the same length—

$$S_m = R_1 - \Sigma W_a - \frac{\Sigma (W_1 x_1)}{a}$$

The total shearing force on the portion will therefore be—

$$S = R_1 a - \Sigma (W_a a) - \Sigma (W_1 x_1)$$

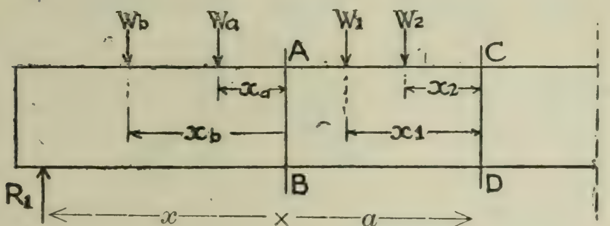


Fig. 72.

Now the bending moment at A B is—

$$B_2 = R_1 x - \Sigma (W_a x_a)$$

and the bending moment at C D is—

$$B_1 = R_1 (x + a) - \Sigma [W_a (x_a + a)] - \Sigma (W_1 x_1)$$

$$= R_1 x + R_1 a - \Sigma (W_a x_a) - \Sigma (W_a a) - \Sigma (W_1 x_1)$$

The increase in bending moment over the length a is evidently—

$$B_1 - B_2 = R_1 a - \Sigma (W_a a_a) - \Sigma (W_1 x_1) = S$$

The calculus proof is as follows :—

∂x is an indefinitely small part of a beam carrying a continuous distributed load w per unit of length, where w is not necessarily constant, but ∂x is sufficiently small to warrant us taking w as constant over the length. Let S and $S + \partial S$ be the shearing forces, M and $M + \partial M$ the moments at either end of the length ∂l as shown in Fig. 73.

Equating upward and downward vertical forces on the length δx :—

$$S+\delta S=S+w\,.\,\delta l$$

$$\delta S=w\,.\,\delta l$$

and

$$\frac{dS}{dl}=w$$

that is to say, the rate of change of shearing force (represented by

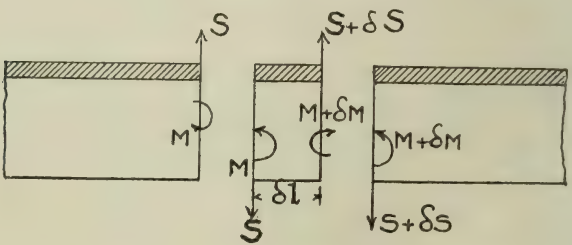


FIG. 73.

the slope of the shearing force curve) is numerically equal to the intensity of loading.

Integrating between two sections $l_1 - l_2$ apart

$$S - S_0 \text{ (the total change in shearing force)} = \int_{l_1}^{l_2} w\,.\,dl$$

or

$$S = S_0 + \int_{l_1}^{l_2} w\,.\,dl$$

taking appropriate signs for each term.

Equating moments of opposite kinds, of all external forces on the piece of length δl, about any point in the left-hand section.

$$M + (S + \delta S)\,\delta l - w\,.\,\delta l \times \frac{dl}{2} = M + \delta M$$

$$\delta M = S\,dl \text{, to the first order of magnitude,}$$

and

$$\frac{dM}{dl} = S$$

That is to say, the rate of change of bending moment is equal to the shearing force. Thus the total change of bending moment from l_1 to l_2 is $\int_{l_2}^{l_1} S\,dl$, which is proportional to the area of the shearing force diagram between the ordinates at l_1 and l_2.

This means that the ordinates of the shearing-force diagram are proportional to the slopes of the bending moment curve. Where the shearing force passes through a zero value and changes sign the value of the bending moment is a maximum or minimum.

The fact that the vertical shear stress at any point is accompanied by a horizontal shear stress of equal intensity, means that the former will tend to produce a vertical relative sliding on either side of the vertical section and the latter will tend to produce a horizontal relative sliding on either side of any horizontal or longitudinal section. The intensity of shear

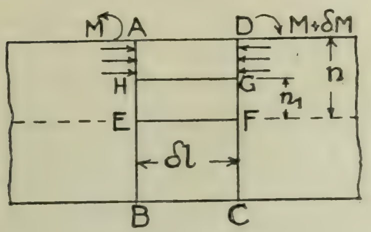


FIG. 74.

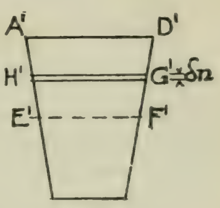


FIG. 75.

stress at a distance n_1 from the neutral axis of a beam may be found approximately thus—

Let A B and C D be two cross-sections of the beam distant δl apart measured along the neutral plane E F (Figs. 74 and 75). Let the variable breadth at any height n_1 from E′ F′ be denoted by b_1. Let the moment at the section A B be M and at C D be $M + \delta M$. Then at any height n_1 from the neutral plane the longitudinal direct stress on section A B is $p = \frac{M n_1}{I}$ where I is the moment of inertia of the cross-section. Consider the equilibrium of a portion A D G H. On any element of cross-section of area $b \cdot \delta n$ the longitudinal thrust at A H is $p \cdot b_1 \cdot \delta n$ or $\frac{M n_1}{I} b_1 \delta n$.

But at D G at the same height the thrust is—

$$\frac{(M + \delta M)}{I} n_1 \cdot b_1 \cdot \delta n$$

The thrusts at D G being in excess of those at A H by the difference in the quantities, namely—

$$\frac{\delta M}{I} \cdot n_1 \cdot b_1 \cdot \delta n$$

the total excess thrust in the area D G over that at A H will be—

$$\int_{n_1}^{n} \frac{d M}{I} \cdot n_1 \cdot b_1 \cdot dn \quad \text{or} \quad \frac{d M}{I} \int_{n_1}^{n} n_1 \cdot b_1 \cdot d n$$

where n is the distance to the extreme fibre, *i.e.* A E, and b_1 represents the variable breadth of section between H′ G′ and E′ F′. Since the nett horizontal force on the portion A D G H is zero, the excess thrust at D G must be balanced by the horizontal shearing force on the surface H G. Therefore if s represents the mean intensity of shear stress at a height n (neglecting any change in s because the length δl is so small), the shearing stress on H G is $s\, b_1\, \delta l$, and

$$s\, b_1\, \delta l = \frac{d M}{I} \int_{n_1}^{n} n_1 \cdot b_1 \cdot d n$$

Hence $$s = \frac{d M}{d l} \cdot \frac{1}{I\, b_1} \int_{n_1}^{n} n_1 \cdot b_1 \cdot d n = \frac{S}{I\, b_1} \int_{n_1}^{n} n_1\, b_1\, d l$$

where $S = \frac{d M}{d l}$ = total shearing force on the cross-section of the beam. Actually the intensity of shear stress at a height n_1 varies somewhat laterally, being greatest at the inside, there being shear drag on the longitudinal vertical sides of each fibre.

In the expression $\frac{S}{I\, b_1} \int_{n_1}^{n} n_1\ b_1\ d\, l$, the symbol b_1 outside the sign of integration and the symbol n_1, which is the lower limit of integration, refer to a particular pair of values corresponding to the height above

the neutral surface for which s is stated, while in the product $n_{\mathrm{I}}\ b_{\mathrm{I}}$ within the sign of integration, each letter refers to a variable over the range n_{I} to n, or H to A.

The quantity $\int_{n_{\mathrm{I}}}^{n} n_{\mathrm{I}}\ b_{\mathrm{I}}\ d\,n$ is the moment of the area A′ D′ H′ G′ about E′ F′, which is equal to the area multiplied by the distance of its centre of gravity or centroid from the neutral axis, or the area of so much of a modulus figure as lies above H′ G′ multiplied by $\mathrm{A\,E} = n$, so that

$$s = \frac{\mathrm{F} \times n}{\mathrm{I} \times \mathrm{H'\,G'}} \times \text{area of modulus figure between } \mathrm{A'\,D'} \text{ and } \mathrm{H'\,G'}$$

This affords a graphical method of calculating the intensity of shear stress at any part of the cross-section.

By calculation in this way the distribution of shear stress in a rectangular isotropic beam is found to be parabolic and of maximum intensity at the neutral axis, there being 1·5 times the mean stress.

In a circular section the stress also varies parabolically, but the maximum stress at the neutral axis is 1·33 times the mean stress.

In beams with thin webs, such as steel joists and plate girders, it is sufficiently accurate to consider the shear stress as of equal horizontal intensity throughout, taken only on the web, so that—

$$s = \frac{\mathrm{S}}{t\,d}$$

where t is the thickness and d the depth of the beam.

It should be noted that shear stresses can occur on three planes, as pointed out for the six components of stress in Appendix IV. Thus the determination by the foregoing method of the shear in horizontal and longitudinal sections whose elevations and plans are the width of the web thickness, while correct in the web of a joist, is not so where the cross-section of the joist changes and becomes not of uniform section. Then the shearing force in the web puts compression and tension into the flanges by dragging against the longitudinal vertical sections one on each side of the web. The rate of change of the compression or tension put into the flanges being the outcome of the total shear drag which is realized on two longitudinal planes, there is evidently variation of shear across the flange transversely and vertically, which results in the principal stresses of tension and compression varying somewhat across the width of the flange, being of greatest intensity at the middle where it adjoins the web.

If the angle of inclination of the principal stresses be calculated at

numerous points on a beam in the manner referred to in Appendix V, one is able to trace curves showing the line of action of such stresses throughout the beam as shown in Fig. 76, which applies to evenly distributed loading. If the beam has a thin web the vertical shear is fairly constant throughout and the horizontal stresses are small, so that the lines of principal stress become inclined at about 45° to the flanges throughout and do not curve appreciably. The diagonal compression tends to cause buckling of the flange considered as a strut. To determine the section of the web the diagonal compression is computed in manner as aforesaid on a diagonal strip and then designed as a long pillar pin-connected at the ends, if the web is deep, or with varying degrees of fixity for less slender

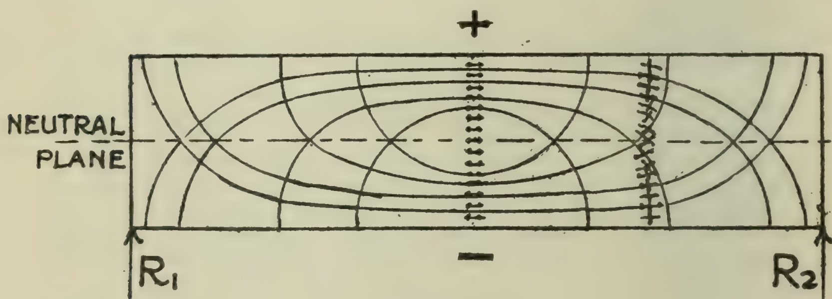


FIG. 76.

webs. On such a basis of analysis, allowing for average conditions of practice, the following rule will apply :—

$$\text{Max. allowable web shear stress} = \frac{\text{Maximum allowable direct stress}}{1 + \dfrac{1}{1250}\dfrac{d^2}{t^2}}$$

where d is the depth of the web and t the thickness.

A travelling load on a structure causes the bending moment, shear, deflection and slope at any given point to change continuously. The variation of any one of these factors may be shown by means of a curve called an "influence line." Referring to the beam shown in Fig. 77, loaded at B by a weight W, the shear influence line for the point A is the diagram below the beam. The shear at A of a unit load placed at B is represented by the ordinate B_1 and the shear at A when the load is placed to the right of A but infinitely close thereto is shown by $\overline{A_2 D}$ while when it is to the left of the point it becomes $\overline{A_1 D}$. As it moves

still further to the left the shear becomes the ordinate to the influence line immediately under the point. The proof is as follows: The shear at any point A is equal to the reaction at O, and for a unit load this reaction is $R_1 = \frac{l-x_1}{l}$. If we plot values R_1 for values of x_1 from x_2 to l we shall have the straight line $\overline{A_2 C_1}$. Similarly for a unit load on the left of A the negative shear will afford the straight line $\overline{A_1 O_1}$. Since the slopes of these lines are equal the lines are parallel. Knowing the value of the ordinate B_1 to the influence line of a unit load the value for a load W will be W B_1.

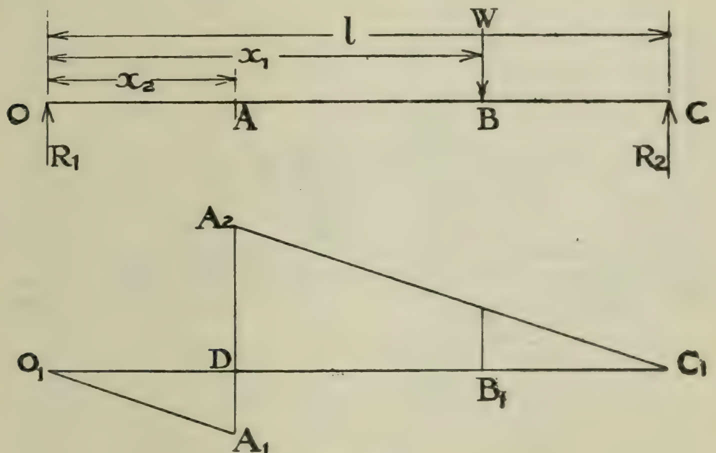


FIG. 77.

A distributed load in this connection may be considered as a series of point loads. If the distribution is even evidently the shear at any point A of a load W_1 extending from say B to C will be equal to the W_1 times the area of the influence diagram from B_1 to C_1.

We now require to develop a relation between several travelling loads and the influence diagram.

It is evident from what has been explained above that if a load be moved to the left it will increase the negative shear until the load passes the point A (see Fig. 78) about which we are determining the shear; consequently the shear becomes a maximum when one of the loads is situated immediately over it. It is again evident that while the loads to the right of A gradually increase the total shearing force when gradually

moved towards the left, those to the left decrease it. Generally the maximum shear at A occurs with few or no loads on the smaller segment.

Now let the distance apart of two consecutive loads, w_1 and w_2, be a, and let them be brought in turn to the point A. Of the total load W call

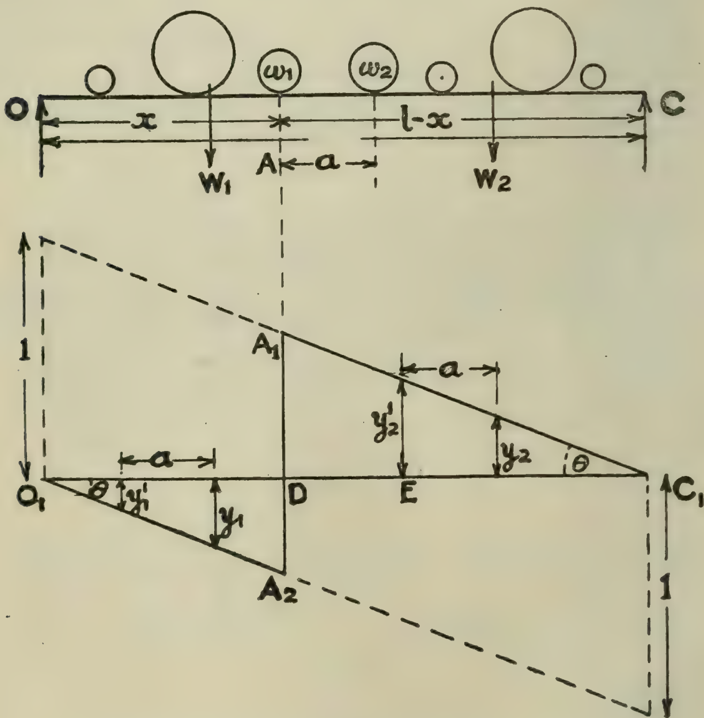


FIG. 78.

the resultant of the load to the left W_1 and the resultant to the right W_2, as before. Then directly w_1 passes A the shear is decreased by an amount w_1, thus becoming

$$W_2 y_2 - W_1 y_1.$$

If the load system continues to move to the left the shearing force is gradually increased until w_2 is infinitely close to A when the shear becomes

$$W_2 y_2' - W_1 y_1'$$

Now

$$\frac{y_2'}{C_{\mathrm{I}}E} = \frac{y_2}{C_{\mathrm{I}}E - a} = \tan\theta = \mathrm{I}/l$$

Therefore

$$y_2 \quad - y_2 = C_{\mathrm{I}}E \tan\theta - (C_{\mathrm{I}}E - a)\tan\theta = a\tan\theta$$

Similarly

$$y_{\mathrm{I}} - y_{\mathrm{I}}' = a\tan\theta$$

The increase in shear is—

$$W_2 y_2' - W_{\mathrm{I}} y_{\mathrm{I}}' - (W_2 y_2 - W_{\mathrm{I}} y_{\mathrm{I}})$$

$$= W_2(y_2' - y_2) + W_{\mathrm{I}}(y_{\mathrm{I}} - y_{\mathrm{I}}')$$

$$= W_2\, a\tan\theta + W_{\mathrm{I}}\, a\tan\theta$$

$$= (W_{\mathrm{I}} + W_2)\, a\tan\theta = W\, a\tan\theta = \frac{W\,a}{l}$$

Hence either w_{I} or w_2 when placed at the point A will cause the shearing force to be a maximum according to whether w_{I} is greater than or less than $W\,a/l$, that is, according as $\frac{w_{\mathrm{I}}}{a} \gtrless \frac{W}{l}$. Thus, to determine the position of the load which will give the maximum shear at any point, work out the value of W/l (*i.e.* the average load for the whole span) and then values of w/a (*i.e.* the average load for the distance between any two contiguous loads). The individual load which affords the greatest difference of the latter value from the former will, when placed on the point, cause the greatest shear at the point.

The way of determining the pitch of rivets uniting the flange to the web of a plate girder is as follows:—Referring to Fig. 79, let S be the shear in the girder at any section, F_h the horizontal shearing force in one rivet, a the arm between the centres of rivet lines, and p the pitch of the rivets.

Taking moments about the top rivet to the right we have—

$$S p = F_h\, a$$

and

$$p = \frac{F_h\, a}{S}$$

If a point load be applied to the flange as shown in the figure there will be the additional vertical punching shear on the rivets. Taking this over the

three rivets we have the vertical force $F_v = S/3$. The resultant of the horizontal and vertical forces may be found in the usual way, being equal to $\sqrt{F_h^2 + F_v^2}$.

The effect of shearing upon a rod projecting from a surface of concrete is to cause a compression of the concrete on the opposite side of the

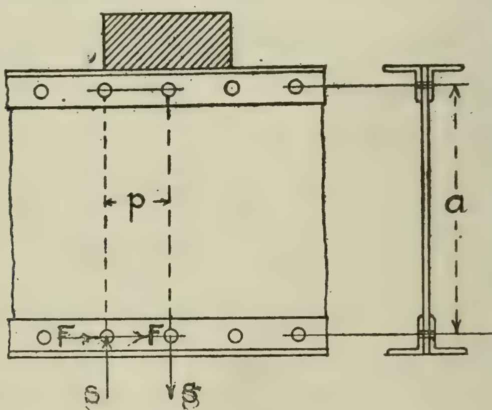


FIG. 79.

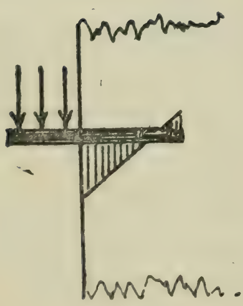


FIG. 80.

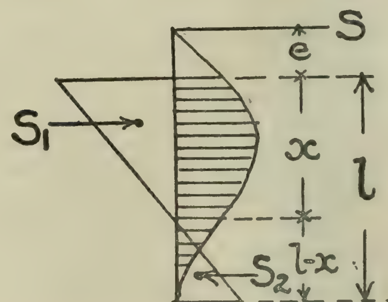


FIG. 81.

rod above and on the same side below as indicated in Fig. 80. The maximum compression occurs at the surface, and since the deflection of the rod is comparatively slight for safe working conditions, the bearing pressure on the concrete may be regarded as varying triangularly as shown.

Let the shear be S, the maximum compressive stress on the concrete c, and e the distance from the point of application of the shearing force

to the face of the concrete, l the length embedded, and x and $l-x$ depths of the concrete in compression on opposite sides of the rod (Fig. 81). The resultants S_1 and S_2 represent the unshaded triangular areas respectively, or $S_1 : S_2 = x^2 : (l-x)^2$. Hence $S_1 = S_2 x^2/(l-x)^2$ For equilibrium the sum of the horizontal forces must equal zero; therefore—

$$S_1 = S + S_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Taking moments about the line of action of P—

$$S_2 \left(l+e-\frac{x}{3}\right) - S_1 \left(e+\frac{x}{3}\right) = 0 \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Substituting the value of S_1 from (1), and reducing, there results—

$$x = \frac{l(2l+3e)}{3(l+2e)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Let the diameter of the rod be d. The resultant S_1 equals $\frac{1}{2} x d c$. Therefore from (1) we get—

$$S = c d l^2/(4l+6e), \text{ or } c = S(4l+6e)/d l^2 \quad . \quad . \quad (4)$$

The bending moment diagram resulting from such loading is also shown in Fig. 81.

The same analysis will of course apply to nails, spikes, wooden pins, and tree-nails. With wood it is necessary to consider the bearing on the side of the fibre for the pin as well as the bearing either on the side or end of the fibres for the wood in which the pin or tree-nail is inserted.

If the rod, bolt, or the like pass between two surfaces subjected to shear the distribution of the pressure on the rod or bolt will be like that of the foregoing unless bending is produced in the rod or bolt for distance e equal to zero, whence $x = \frac{2}{3} l$, l being the length on one side of the surface of separation.

In built-up timber beams hardwood keys are often inserted as shown in Fig. 82. The shearing force upon the ends of the key is fairly evenly distributed, but upon the sides the pressure will vary triangularly, the maximum pressure being at the end, as shown in Fig. 83. The tendency the key has to rotate will cause the timbers to separate unless they are held together by bolts. The tension in the bolt is therefore directly related to the pressure on the sides of the key.

The length required to resist longitudinal shearing of the fibres of the key is evidently $l = \dfrac{S}{s\, b\, l}$.

The depth of key required will be $d = \dfrac{S}{b\,l\,c_1}$ where c_c is the compressive resistance of the end fibres.

The moment of rotation due to eccentric compression at the ends of the key is $T = \dfrac{S\,d}{2}$.

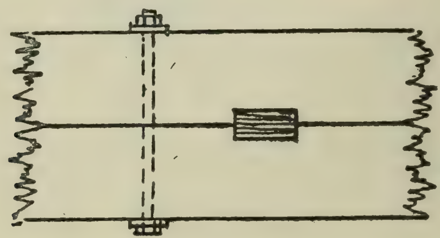


FIG. 82.

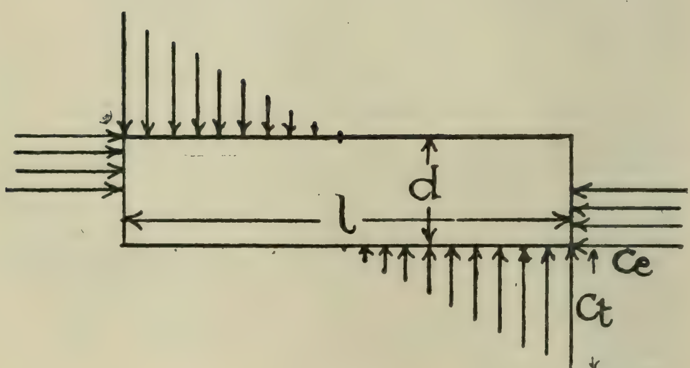


FIG. 83.

The moment of resistance is $R = \dfrac{c_t\,b\,l^2}{6}$ where c_t is the compressive stress transverse to the grain. As $T = R$ we possess all the data for seeing that the stress is not excessive.

The determination of the stresses on metal pins or bolts in timber where the fibres have unequal resistance to endwise and transverse compression is dealt with in Professor Henry S. Jacoby's "Structural Details" (Chapman and Hall).

DISCUSSION.

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., etc.) :—It is possible that this paper may be considered rather abstruse by some of our members, but the subject is one of very great importance and well worth prolonged study. Structural engineering is now becoming a very complex matter, and it is absolutely necessary that you should have a sound basis of theory in order to make solid progress in it. I do not propose to enter into the discussion now, but I have here a small indiarubber model of a rectangular beam that I think will be of interest to you in connection with this subject at the present moment. One side is plain, another side is marked with transverse parallel lines $\frac{1}{4}$ in. apart, another side is marked in $\frac{3}{8}$-in. squares, and the remaining side is marked out in $\frac{1}{2}$-in. circles, touching each other. This can be easily stretched or bent or compressed or twisted, and the alteration in the shape of the various figures shows you exactly what is happening to the fibres and the nature of the stresses. It also shows you the alteration produced in the shape of the cross-section when the beam is bent, hollow there, bulging here, etc., and it is astonishing what a number of interesting experiments can be made with it. Among other things you will notice that it is very easy to detect the position of the neutral axis upon it when the beam is bent. You will see that the parallel lines are farther apart on one edge, closer together on the other edge, and of unaltered distance at the centre.

MR. CHARLES F. MARSH, M.Inst.C.E., agreed with Mr. Dyson's remarks with respect to the shearing stresses in fixed beams and continuous beams on page 87 of his paper. With regard to the spacing of stirrups (pages 94-96), Mr. Marsh did not agree that the inclination of the diagonals should always be 45 degrees, but favoured a considerable play in the spacing of these members, as it was convenient to incline them at varying angles to allow the proper resistance to varying shearing stress. With respect to the vertical shear members, he thought it more

economical, safer, and more convenient to space them at varying distances apart, keeping them of the same sectional area, than to keep them the same distance apart and vary their sizes and number of branches. With regard to the allowable stress (on page 97), he advocated that the allowable stress on diagonal tension members should be three-quarters that in direct tension, because of the possibility that the actual stress might be in excess of those calculated and not because these members were considered as acting in shear. Mr. Dyson's paper appeared to bear out this feature, as, for instance, on page 79, from "the mathematics, however, are strictly exact" to "it is well to adopt it for simplicity." With which Mr. Marsh agreed, but pointed out that Mr. Dyson admitted that the results may not be actually accurate. Also on the same page from "even if we take the distribution" to "greater as the shear stress increases." Further, on page 79, from "In Mr. Prichard's paper" to "something very considerable," as to which Mr. Marsh remarked that with a short beam it would be very considerable, and as the beam became longer it would probably be less. While on page 98 the same point is made from "It should be borne in mind" to "modify the former theory." The next point Mr. Marsh referred to was in regard to Mr. Dyson's remarks as to inclined bars, on page 97, from "If bars are inclined at a flatter angle" to "obtaining $\frac{S \operatorname{cosec} \theta}{2}$ as the tension." He pointed out that this indicates that up to 45 degrees Mr. Dyson allows half the diagonal tension stresses to be taken on the concrete and half on the steel, but directly he gets any angle less than 45 degrees to the horizontal he then considers that the whole of the diagonal tension stresses come on the steel. Mr. Marsh considered this unjustifiable, as at 45 degrees half was allowed, while at 44 degrees 59 minutes the whole of the diagonal tension stresses must be taken by the steel. He suggested some angle less than 45 degrees, say 25 degrees, should be assumed as that at which the steel took the whole of the stress, and between the 45 and the 25 the concrete should be assumed to take less and the steel more, on a straight line assump-

tion. At 45 degrees it would take one-half, and a straight line proportion between 45 and 25 degrees where it would take the whole.

DR. OSCAR FABER, D.Sc., Assoc.M.Inst.C.E., A.M.I.E.E., A.C.G.I. (Member of Council C.I.), showed a number of lantern-slide views of reinforced concrete beams which he had made and tested and illustrating his theory of inclined compressions which was subsequently set out in detail in a paper that he read before the Concrete Institute on May 18, 1916. As this paper is printed *in extenso* in the TRANSACTIONS, his remarks in the discussion on Mr. Dyson's paper are not printed.

MR. ETCHELLS, F.Phys.Soc., A.M.Inst.C.E. (Member of Council C.I.) :—With regard to the question of cast iron and concrete crushing at 30 degrees to the axis of the compressive stress, that ratio has been used a very great deal for foundation works where the spread of the concrete has been such that the angle of dispersion has not been greater than 30 degrees. In this case no reinforcement has been put in at all, and when the foundation has been taken out years after it has been quite satisfactory. That 30 degrees is of course measured from the vertical axis.

The author says: "It is well known that webs need to be stiffened by the provision of vertical stiffeners under point loads to prevent crushing and buckling of the web." As to the calculations of web thickness necessary there are marked differences of opinion. Some designers are inclined to design the web to resist buckling at any part, but there is a newer school which is trying, and with good success, to put in the stiffeners as though they were struts of a lattice girder and leave the web of a nominal thickness of $\frac{1}{2}$ in., and consider it merely as a tensile diagonal member, of which only the central portions are in effective use. When this is done there is no need for the intricate calculations of web resistance, and in addition there is economy in the design, combined with safety, because then vertical compressions are taken by vertical stiffeners and diagonal tensions

by the more central portions of the web. That is one of the practical applications of a very theoretical theory.

With regard to page 92, Mr. Dyson says: "Seeing that if there should be cracks in the beam extending practically to the neutral axis, obviously there can be no such horizontal shear stress across the cracks, and if there is no horizontal shear stress therefore vertical shear stress cannot exist there either." The practical conclusion to be drawn from that is that it is illogical to calculate the whole of the web area as being of effective use in shear. Surely only the compressed portion of the web can be taken as in shear; but remember that if we take only the compressed portion of the web, the permissible shearing resistance per square inch could be logically and safely increased.

FIFTY-THIRD ORDINARY GENERAL MEETING

THURSDAY, DECEMBER 17, 1914

THE FIFTY-THIRD ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, December 17, 1914, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I. Mech.E., F.S.I., M.S.A. (the President), in the Chair.

The following were elected :—

ASSOCIATE-MEMBERSHIP.

JOHN MARSHALL RODGER, Assoc.M.Inst.C.E., Harbour Engineer's Office, Greenock, N.B.

HUBERT VICTOR STEPHENSON, Quantity Surveyor, Westminster, S.W.

STUDENTSHIP.

REGINALD BOYD, Westminster.
FRED SEFTON CROWTHER, Westminster.
FREDERICK SEARLE CURRAN, Westminster.
THOMAS HENRY HALL, Westminster.
FREDERICK RINGS, Jun., Westminster.
CHARLES WILLIAM WEST, Westminster.

DISCUSSION.

THE PRESIDENT :—We now have to resume the discussion on Mr. Dyson's paper.

MR. H. KEMPTON DYSON :—Since the last meeting I have taken the opportunity of making a number of corrections in the paper.

Mr. S. Bylander, M.C.I. :—In regard to the assumptions for calculating shear reinforcement, I notice that Mr. Dyson suggests that when the bars are bent up to an angle of 45 degrees, or where the shear reinforcement has an angle of 45 degrees to the axis of the member, half the amount of shear may be calculated as taken by the steel, while the other half is taken by the concrete in compression. With that I agree. Further, Mr. Dyson suggests that the stirrups or web reinforcement when placed vertically should be calculated to take the whole of the shear in the effective depth of the beam. To this I also agree. I would suggest that Mr. Dyson should amend his recommendation that for shear reinforcement having an angle of less than 45 degrees with an axis down to an angle of 30 degrees the amount taken by the steel should be proportionate from 45 to 30, and that at an angle of 30 degrees the whole of the shear should be taken by the steel *or* concrete, and when it is vertical the whole of the shear should be taken by the steel only, at angles between these limits proportionately from whole when vertical shear members and half when sloping 45 degrees. I was very interested to see and to listen to the explanations by Mr. Faber at the last meeting. I think the demonstrations of the behaviour of short beams on account of shear were excellent. I thoroughly agree with his theory that we should calculate the concrete in compression in such a case. I studied the question about a year ago in connection with the design of reinforced concrete footings used for supporting steel pillars. I then made the assumption that the vertical section of the reinforced footings should be considered as a triangle, one side being the base and the other two sides being the slope of the concrete, the load being upheld at the apex of the triangle. It is obvious that a big block of concrete can resist shear in quite a different way from a narrow and long beam, and therefore we have two extreme cases: very short beams and very long beams, which necessarily must be treated differently. The one weak point about this theory is with regard to the adhesion or fixing of the ends of the reinforcement. As Mr. Faber pointed out, the beams had failed very often on account

of the straightening of the hooks. In connection with reinforcement for ordinary beams, I submit that the spacing of the vertical stirrups should be sufficiently close and that the sizes should be such that the anchorage at the ends of the stirrups should be sufficient to transmit the stress to the concrete ; that is to say, that the spacing should not exceed, say, half the effective depth in order to obtain the result we assume. The reason why failure has not occurred on account of ends of stirrups not being fixed I put down to this, that the full stress has not been placed on the stirrups ; the concrete has taken a considerable portion without putting the stress on the stirrups. I will add that a limit should be placed on the amount of shear reinforcement to be used ; that is to say, the area of the cross-section of the concrete of the beam should not be stressed beyond a certain limit : I would say 120 to 150 lb. per square inch, irrespective of how many vertical shear members there may be, as bonding may not be sufficient.

MR. W. A. GREEN, M.A., B.Sc.Eng., Assoc.M. Inst.C.E., M.C.I., after congratulating the Concrete Institute on having a member able to devote so much time in producing such a valuable paper, referred first to the question of shear in the webs of joists, and showed with a proof a graphical solution for the principal stresses, which will be found illustrated in *Engineering News*, vol. 71, p. 977.

In applying this theory to the web of a 15 in. $\times$ 5 in. B.S.B. stressed up to the Building Acts limits, he showed that the principal compressive stress at the fillet was 8·8 tons per square inch, and the principal shear stress at the neutral axis 6·36 tons per square inch, which he regarded as unduly high.

He then gave a graphical comparison of various formulæ for permissible shear, including one derived from Professor Fidler's column formula for fixed ends, assuming that the tension normal to the compression in the web has a reinforcing effect equivalent to that of halving the effective strut length of the compressed web, measured at an angle of 45 degrees with the neutral axis. This comparison he hopes to embody in a future contribution to the Institute.

With regard to the author's ingenuity in deriving a theory fitting assumptions and experimental data, he referred to a story of the late Sir Benjamin Baker which is recorded in the Minutes of the *Proceedings of the Inst. C.E.*, vol. 181, pp. 227, 228.

Mr. Green, in conclusion, attempted to controvert the assertion that it is impossible to take shear in the lower portion of a reinforced concrete beam, pointing out that sliding was impossible along the cracks in the concrete unless the reinforcement slipped through the concrete and allowed the cracks to open.

MR. EWART S. ANDREWS, B.Sc.Eng.:—I want to make my text the remark which you will find on page 70, at about the middle: "Brittle materials are weak in tension, and the author of this paper offers the opinion that rupture of ductile materials occurs by shearing, but that rupture of brittle materials occurs by tension." I believe that that is not only the opinion of the author of this paper, but the opinion of the best authorities on the subject, and I think it has been proved almost without the shadow of a doubt that that is correct. But what I think have not been realized are the effects that follow from that statement. If it is correct, it is nonsense to speak of a shear failure of concrete; it is always a tension failure, and when we are speaking of what we commonly call shear failure it is merely a tension failure at the point which is the point of maximum shearing force as opposed to the point of maximum bending moment. If it is true that in ductile materials such as steel failure is always by shear, and if you regard the elastic limit as the criterion of failure, the shear strength of steel is half the tensile strength, and therefore if $7\frac{1}{2}$ tons is the safe tensile strength of steel, $3\frac{3}{4}$ tons is the safe shear strength. If 5 is the safe shear stress, 10 is the safe tensile stress. You can have it whichever way you like, but the relation between the safe pure shear and the safe pure tension is as 2 to 1. It follows from the equation for Guest's law that if pure shear is nothing you are left with a pure tension. If you are left with a pure tension, your equation gives you that the shear equivalent to a pure tension is half that tension. It is called Guest's

law because Guest was the first experimenter to really prove it. I think it is also called Tresca's formula, and it has been proved absolutely up to the hilt. Turner found that the calculated elastic limit stress in torsion was exactly half that found in tension. That, then, is the result as far as steel is concerned of this question of the equivalent shearing stress. In regard to the "gospel of the yield point," I think that is an attack which ought to be met. "The reliance in practice on such joints" (Mr. Dyson is speaking of the riveted joints that after riveting some rivets come into play before others) "would seem to be overlooked by engineers who have accepted as gospel the modern idea that the factor of safety should be determined, not upon the ultimate resistance, but upon the yield point." All I can say is that if we do not base our safe stresses upon the yield point, as Mr. Dyson calls it, or the elastic limit, as I think he ought to call it, it is perfectly useless to use the ordinary bending formula $B = f M$. If we use that formula at all, it is useless to do anything else than to work our safe stresses on the elastic limit, because otherwise the term "factor of safety" is meaningless. Mr. Green has referred to the fact that when you test beams to destruction the calculated stress does not agree with the tensile tests, and that is, of course, due to the reason that the assumptions that you make in developing your beam formula have gone directly you exceed the elastic limit in the case of steel, or in the case of cast iron have gone at the beginning, because the stress is never proportional to the strain. Unless your stress is proportional to the strain—i.e. unless your stresses are based upon the elastic limit—these formulæ are meaningless.

I am sorry to disagree with Mr. Bylander as to the part of the shear in a reinforced concrete beam which may be regarded as taken by the steel. As far as I can understand it, if the steel takes anything, it must take the lot, and that is the beginning and end of it. The assumption that your steel has got up to a diagonal tension stress of 16,000 or 12,000 lb. per square inch shows that your concrete has absolutely gone out of action and that it is essential to neglect it. The only reasonable way that you could take part of the

shear in the steel and part in the concrete is, I think, the way which I believe Mr. Faber first indicated in his book, and that would be to restrict your stress in the steel to 15 times the safe tensile stress in the concrete, which is 900 lb. per square inch. The principal point that I want to make to-night with regard to this is that the stresses in a beam of reinforced concrete or any other beam—the resultant effective stresses at any point vary right across the section. When you speak of compression down a particular line, it is merely a convenient way of dealing with it. If you take any particular line, you will have stresses varying right across that line. It is very difficult to get an exact idea of the stresses on the short-cut methods without being liable to get very much beyond the facts of the case.

Now we come to the consideration of a beam and the so-called truss action. Mr. Faber has got some extremely interesting experiments which show that when fracture actually occurred in that beam—a particularly short beam for its depth, a beam to which it is well known and stated emphatically in every good text-book you cannot apply the ordinary bending formula and expect to get the correct results for the stresses—the failure was by a kind of truss action. The fact of the matter is that directly you start getting failure of one kind you alter entirely the nature of your structure. That would start acting as a beam until the tensile stress in the reinforcement, as it were, soaked outwards to the end until it got constant, and when the tensile strength becomes constant you get the pure truss action. But that, of course, demonstrates the fact that when your beam has ceased to be a beam it has become something else. It does not show that so long as a beam is acting as a beam the ordinary theories of beam stresses are incorrect. You must make up your minds to start with whether or not you will base your calculations and your formulæ upon theories which are based upon the laws of elasticity. If you are going to do that, then you must work your working stresses on the elastic limit. If you are not going to do that, you are going to go entirely by breaking stresses, and then the only scientific way of doing it is to

throw over beam formulæ altogether and to have a grand and glorious set of tests made and deduce purely empirical laws from those tests. My own opinion is that the more you attempt to deduce empirical laws from tests the greater becomes the muddle. You have to remember that to make tests accurately requires a very particular and special kind of skill, very good and accurate apparatus, and that it costs a tremendous amount of money.

Mr. CYRIL COCKING, M.C.I., next took part in the discussion.

Mr. R. W. VAWDREY, B.A., Assoc.M.Inst.C.E. (Member of Council C.I.) :—I think this is the most important paper that has ever appeared before the Concrete Institute, and to attempt to criticize the whole of it in detail after reading it for perhaps three hours, when Mr. Dyson says he spent three years on it, would be an impertinence, but there are one or two points to which I should like to call his attention. On page 65 I appreciate very much indeed the remarks he makes in general on the comparison between theory and practice. The whole of that paragraph is well worth reading carefully and is capable of being appreciated by everybody. On page 70 one of the previous speakers remarked on the following being a very important sentence indeed : "Brittle materials are weak in tension, and the author of this paper offers the opinion that rupture of ductile materials occurs by shearing, but that rupture of brittle materials occurs by tension." I should like Mr. Dyson to add to that a careful definition of what failure in tension and failure in shearing is. It appears to me that there is a good deal of opening for misconception unless he adds that. Just to make that statement about failure by shearing and failure by tension seems to me slightly to beg the question, without giving a careful definition. On page 71 the author says : "The manner in which a metal yields when it takes any kind of permanent set is by slips occurring on cleavage of gliding surfaces within each of the crystalline grains." Now, what prevents that slipping, if it has once begun, from continuing? If movement actually takes place between the particles,

perhaps Mr. Dyson will explain what brings them to rest. I gathered that Mr. Faber considered that his remarks were in opposition to those of Mr. Dyson, and I presume that perhaps Mr. Dyson considered that his remarks were in opposition to those of Mr. Faber, but as far as I am concerned it does not appear to me that there is the slightest antagonism between the two points of view. Mr. Faber's beams were designed so that they could only act, when tested to failure, as struts with a tie across the bottom. Mr. Dyson's remarks, of course, apply to an absolutely different type of beam which will fail in an entirely different way, and the fact that Mr. Faber's beams acted as struts which failed under compression does not appear to me to militate in the least against the argument brought forward by Mr. Dyson from the conclusions which he reaches. Undoubtedly, to my mind, it is in the case of short beams, particularly (as has been mentioned by others) column footings —the most ordinary case where you get a very heavy load on a short beam—it is in that case that the strut and tie action mentioned by Mr. Faber will occur, and there is not the faintest doubt in my mind that the only way of dealing with these problems is on the lines mentioned by Mr. Faber; but that does not in any way militate against Mr. Dyson's remarks.

On page 90 Mr. Dyson mentions the question of the elastic limit of the steel in rivets. I feel that I appreciate Mr. Dyson's point, but it does not appear to me to be at all the point which I think Mr. Andrews mentioned. I agree that if the ultimate point of failure is being considered, obviously the ordinary formulæ do not apply in the least degree. It is stated in every text-book that the formulæ for bending only apply so long as the elastic limit has not been passed, but that is not Mr. Dyson's point, surely. I think he says that engineers who say that under no circumstances must one consider the use of steel up to the elastic limit seem to forget that in riveted steelwork of any sort you are probably using a good deal of your steel at the yield point.

I do not follow on page 95 the disadvantage in spacing web members far apart. That is all very well if you are speaking of an open lattice girder,

but in a concrete beam the compression portion of the beam is surely supported on the rest of the beam. There is no portion of the compression member of a concrete beam which is itself acting as a beam from one web member to the other.

On the top of page 98 Mr. Dyson makes a very important point indeed, because it deals with a very ordinary fallacy, and that is that steel reinforcement in a reinforced concrete beam where it crosses at right angles the plane of shear is thought by some people to resist the shear. Of course that is absolutely nonsense. You cannot possibly resist shear across a plane by the insertion of a steel rod normal to that plane. The steel would be in the position of nails between two planks, and would, of course, crush the concrete sideways before it tended to shear. That seems perfectly obvious, but at the same time it is a fallacy which many people hold.

I have spent some time in reading Appendix 7, and I should like to reserve judgment about that. It seems to me an exceedingly interesting suggestion on Mr. Dyson's part, but I do not quite know whether I perhaps realize all that it leads to at present.

MR. ALLAN GRAHAM, A.R.I.B.A. :—Many of the speakers in stating their views on shear have referred to one or other aspect of it. Mr. Faber stated that some of the failures in the extremely interesting experiments shown by him are by compression. Probably the original force that caused the failure was compression, but actually the failure was caused by secondary stresses, induced by the compressive force, so that most of these failures referred to as compressional failures are really the result of diagonal tension. In fact, our bugbear in this subject is diagonal tension, believing as I do that most of the shearing forces can be resolved into diagonal tension. A present-day fancy is that when anything goes wildly wrong we need a so-called practical man, but that is not so. What we need is a practical theorist. A practical man means a man accustomed to mere daily practice in the way things commonly work, the principles of which have been laid out for him by thoughtful theorists in the past. When things will

not work it is to the thinker, who tries to evaluate laws and gives us reasons why they work at all, that we go. Many so-called practical men run down the theorists, yet they are using formulæ every day that they do not understand, and so even they are indebted to the theorists they decry for the knowledge they have. I know something of the immense amount of time and care that Mr. Dyson has given to this paper, and it has taken a long time to elaborate. In fact, he is still evolving, and no doubt he will come to such a conclusion that divergent views will be satisfied and reconciled. It is valuable for a Society to take up such a subject as this and attempt to improve our understanding of it. If we have some reasonable theory of the way in which stresses act, we are in a position to intelligently and economically fashion the various members, and in the case of reinforced concrete beams we can place the reinforcements in the most advantageous positions to meet the stresses. Otherwise in the event of running up against an unusual problem even the so-called practical man has to call on some white-haired, broken-down laboratory professor to analyse the trouble and point out how it is to be overcome.

MR. R. W. VAWDREY :—It may be of interest to know that "the practical man is one who practises the errors of his forefathers." That seems to me to be an excellent definition.

DR. OSCAR FABER and MR. W. G. PERKINS (Member of Council C.I.), then spoke.

MR. H. KEMPTON DYSON :—I am very greatly indebted to the meetings for the kind way in which the paper has been received. I am delighted that my paper should have led to so informative and useful a discussion, and any trouble that I have been put to has been well repaid. With regard to the discussion generally, I think it has been chiefly supplementary to the paper and really hardly tended to controvert the majority of the views which I have put forward. After all, as I said at the outset, I have only selected certain aspects for consideration, and,

notwithstanding the considerable length of the paper, I found it impossible to give examples of practical calculations. Indeed, I rather thought that as the majority of the members of the Concrete Institute were specialists and engineers it would be somewhat intrusive to treat them as elementary students who needed to be taught how to apply theory in practical design, especially as in designing one has to use one's own judgment and persons hold different views. I have referred in a definite way to some of my own methods of employing my own theories in practice. Mr. Marsh stated that he sometimes preferred a different solution from mine, as, for instance, in regard to the kind and spacing of shear members in reinforced concrete beams. I should not think I was entitled to assert that my methods were better than or even so good as his, though I prefer them and have furnished my reasons for so doing. I thought I had explained my views in that respect, when I referred to the cracks that appeared in reinforced concrete following the lines of the principal stresses and appearing to be the outcome of diagonal tensile stresses. As shown in Appendix 8, the cracks are likely to take a fairly vertical position about the middle of the beam and become inclined more at an angle of 45 degrees towards the ends where the maximum shear usually occurs. Cracks occurring in such a way never become flatter than 45 degrees. It seemed to me only reasonable not to ordinarily count upon anything flatter, though I quite agree with Mr. Faber's remarks about the inclined compression struts that are developed from a point load to the support and may incline at a very flat angle according to the ratio of the depth of the beam to the distance of the point load from the support. I consider that if inspection be made of the very careful records by Professor C. Bach, of Stuttgart, and Professor Arthur N. Talbot, of Illinois, of the numerous tests that they have made on beams reinforced with all sorts of shear members and kinds and arrangements of reinforcement, one is compelled to recognize that, following the lines of principal stresses, cracks develop in the concrete due to the tensile stresses which are more or less inclined according to the relative values of the flexual direct

and vertical tangential stresses. It was pointed out in the paper that such diagonal cracks in no wise precluded the concrete from taking diagonal compression parallel to such cracks, but obviously it cannot take place across them any more than horizontal shearing stresses can, the erroneous theory of which I mentioned. Mr. Green, I think, misunderstood my remarks. I do not say that you cannot get a certain amount of shear in the portions between cracks, but the very presence of cracks, I think, nullifies his contention that such resistance is considerable, because the concrete cannot crack unless it has slipped along the bar, and if it has done that you cannot communicate any more into those cracked pieces than the grip allows you to take out of the tension in the bar. If any one refers to recent tests on adhesion in beams—those published at Illinois—they will see that the stress gradually creeps along the bar, as it were, the bar gradually drawing through the concrete. It is that which causes the cracks. My remarks were not to the effect that you could not get some shear in the cracked sections, but that the theory for the calculation of the distribution of stress in which that action is assumed as all sufficient was illogical when you compared it with the other part of the theory where it was assumed that the concrete could and would crack. It was also stated in the paper that the resistance to the shearing force of a beam must, in the presence of such cracks, be altered to the manner in which an arch, a truss, or a frame resists shearing force, and I therefore find myself in complete agreement with Mr. Faber's interpretation of his tests, photographs of which he showed upon the screen. I place the highest value on Mr. Faber's investigation, and I have since the previous meeting had an opportunity of discussing matters with him, and he has given me fuller particulars of his tests. I hope he will shortly furnish this Institute with a paper upon his researches and his method of analysis. Indeed, arising out of this paper and this discussion, several members could render great service by contributing to the elucidation of this elusive subject. I ventured the remark in the paper that by attention to the design it was possible to put the web members very far apart and yet at the same time ensure frame

action. Mr. Faber has shown in detail how this can be done. In the ordinary way, however, the tests of Professor Bach in particular seem to me to show conclusively that the cracks in beams under evenly distributed loading will be fairly vertical and close together about the middle, so that if web members are required thereabout they would be preferably placed vertical and the inclined compression members assumed to run from stirrup to stirrup and not across several stirrups. Nearer the ends, where the greatest shear occurs, the cracks become more inclined, and it appears to me to be the best to go in for the diagonal trussing that I advocated in my paper. Of course, all that I have said in the foregoing does not mean that we cannot resist the shear in part by the vertical component of tension in bars inclined at flat angles like those in a trussed timber beam, but for the reasons given I consider the vertical components should not be taken as more than T sin θ in such cases. As regards the form of shear members, referred to by Mr. Marsh, I gave my reasons for my practice, but when I remarked about different sizes I may say that I have never found occasion to vary the diameter of stirrups in the same beam. It has been my unfortunate experience to find more difficulty in getting variably spaced shear members properly spaced than in getting different kinds of evenly spaced stirrups and cranked bars properly arranged. It seems impossible to get the British working man to understand variable spacing, with the result that he lets the steel go anyhow, but if the members are evenly spaced he seems to see some reason in keeping them so. Of course, all steel ought to be most carefully inspected and approved before concreting is commenced in any part of the work. Mr. Vawdrey asks for a definition of what I mean by the words "tension and shear." I have used them in the sense of normal and tangential forces respectively. Mr. Vawdrey also asks for my view of the reason why slips on crystal cleavage planes come to rest at all. That is a property of matter yet to be elucidated ; there seems evident some powerful attractive force in atoms or molecules directionally arranged.

FIFTY-FOURTH ORDINARY GENERAL MEETING

THURSDAY, JANUARY 7, 1915

THE FIFTY-FOURTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, January 7, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I. Mech.E., F.S.I., M.S.A. (President), in the Chair.

MR. HENRY J. TINGLE, M.Inst.C.E., M.C.I., read his paper, which was illustrated by limelight views, as follows :—

THE APPLICATION OF CONCRETE IN MODERN SANITATION.

THE term "Sanitation" is capable of a wide interpretation, but for the present purpose it is proposed to limit its definition to Works of Water Supply, Main Sewerage, and Sewage Disposal.

In looking through the volumes of TRANSACTIONS OF THE CONCRETE INSTITUTE there does not appear to be specific reference to the use of concrete in sanitation. This omission is probably due to the fact that the Insiitute has only been in existence since 1908, and attention has naturally been concentrated upon the application of concrete to works of greater magnitude, presenting more intricate problems than is the case with the majority of works of sanitation.

Concrete, however, has played its part in sanitation for many years, with more or less success, according to the skill of the designer and the quality of the materials and workmanship. The tendency in recent years has been to employ concrete more and more, and this tendency in the future is certain to extend its use in directions where now it is scarcely applied. Therefore it is not surprising that the Secretary of the Institute should desire recognition to

be given to the claims of concrete in connection with sanitation, however much his judgment may be at fault in his selection of the author of the paper.

The President of the Institute, in his address at the opening meeting of this session, states : "The Local Government Board are responsible for retarding the progress of reinforced concrete to a very great extent."

There is no doubt that sanitation works have been peculiarly affected by the attitude of the Board towards reinforced concrete, as such works are generally undertaken by local authorities, who have to obtain the sanction of the Board to the raising of a loan for the amount of capital expenditure, or else to obtain Parliamentary powers by means of a Private Act. The great majority of authorities, however, adopt the first of these two courses of procedure, and the practice of the Board is, in the case of reinforced concrete for works in contact with water or sewage, to require the capital sum to be paid off in a period of ten years, whilst in the case of ordinary concrete, under the same conditions, the period of repayment is thirty years. In cases where in addition to reinforced concrete in contact with water, there is reinforced concrete not in contact with water, the periods for the loans on each class are sometimes equated together, and the loan for the whole granted for the period given by the equation.

Doubtless in the early years of reinforced concrete the Board were well advised to act cautiously in the matter of sanctioning loans for the use of a combination of materials of which, in this country, there had been scarcely any experience, either as to its suitability, or as to its life in sanitation works. The effect of this differentiation on the part of the Board in the treatment of concrete, and of reinforced concrete, has been to curtail the employment of the latter in public works of sanitation. It is clear that under the present conditions, where a loan is obtained for reinforced concrete, the burden thrown on the ratepayers annually to repay the principal and interest in equal annual instalments spread over ten years, would be more than twice as great as for a loan of equal amount and rate of interest obtained for ordinary concrete, to be repaid in equal annual instalments in thirty years. The following example will illustrate the matter. If £1,000 be borrowed at $3\frac{3}{4}$ per cent. interest to be repaid in equal half-yearly instalments of principal and interest for terms of—

10 years : the amount to be paid each half-year = £60 8s. 6d.
30 " " " " " " = £27 18s. 1d.

The difference between these two amounts is approximately £32 10s., which is 116 per cent. of £28.

Under these circumstances it is only natural that most authorities elect to use ordinary concrete in order to ease the burden on the present ratepayers, and throw part of it upon the ratepayers of the future.

The effect of the differentiation amounts to this, that unless the cost of the works can be considerably reduced by the employment of reinforced concrete as compared to ordinary concrete, there is but little probability of local authorities deciding in favour of the former. The requisite saving may be effected in some instances, but so far as the experience of the author goes this is not generally the case.

With the experience that has now been acquired relating to the behaviour of reinforced concrete that is constantly in contact either with water or with sewage, it is permissible to hope that the time is not far distant when the Local Government Board will reconsider its attitude towards this material, and in cases where it is proposed to make use of it will sanction loans on more favourable terms as to the period of repayment than those now in force.

It is generally agreed that concrete should be machine-mixed. On the other hand it cannot be denied that very good work has been done in the past with concrete hand-mixed, and with the cement calcined in the old type of kiln, and not nearly so finely ground as is the case at present.

In many instances sanitation works are constructed for small populations. Such works are necessarily not extensive, the quantity of concrete required being limited, and a considerable proportion of the whole being laid in thin layers under pipes and manholes along miles of streets. These conditions militate against the use of machine mixers, and the mixing is done by hand, usually by ordinary labourers, necessitating most careful supervision on the part of the inspector.

In the construction of reservoirs and sewage disposal works for larger populations machine mixers are frequently used. They are now being employed by Mr. S. S. Grimley, M.Inst.C.E., at Hendon, on the extension of the local Council's sewage disposal works, the quantity of concrete being 10,000 cubic yards.

Concrete has been and now is extensively employed in the construction of reservoirs, sewage tanks, filters, aqueducts, and water towers. Full descriptions of these works are to be found in the technical journals and volumes of proceedings of the various engineering societies. The same

holds good for similar structures in reinforced concrete, though to a more limited extent, but numerous examples are given in the catalogues issued by the various firms specializing in this class of work.

Mass concrete has been largely used in the construction of sewers of 4 ft. diameter and upwards. It is moulded in the trench, and supported by centering until set. Elliptical sewers are also constructed in this manner. In recent years reinforced concrete has been employed for sewers of both types, the mixture being sufficiently stiff to admit of ramming.

Concrete tubes have been extensively employed in main sewerage work for many years past, being supplied ready for use by firms who have taken up their manufacture.

In Volume VI of the *Proceedings of the Municipal and Sanitary Engineers*, published in 1880, are two interesting papers on the manufacture of concrete tubes, by Mr. C. Fletcher Woods and Mr. J. W. Butler respectively, from which it appears that concrete tubes were introduced in the United States about the year 1845 and into the United Kingdom in 1849, although their manufacture on a commercial scale was delayed in this country until 1875.

The method of manufacture as described in the above papers is very similar to the present. Several tables of tests made for resistance to crushing are given, the following extract being an instance :—

Experiments at Hove, by Mr. E. B. Ellice-Clark.

	Breaking weight in lb.
24-in. rock concrete tubes, 250 lb. in weight 1⅝ in. thickness, 2 ft. long, bedded in clay up to and above the springing	4,938
	4,121
	4,121
Mean ...	4,393

It is also pointed out that concrete made with broken vitreous stoneware, or the waste of stoneware pipe works, is much stronger than that made from gravel. The means of a series of blocks tested for tensile resistance being 161·6 lb. per sq. in. for washed pea gravel and 210·6 lb. per sq. in. for broken stoneware. The mixture in each case was 2 of aggregate, 1 washed sand, 1 Portland cement. Age of blocks, 5 weeks. Attention is also drawn to the fact that several wells had been successfully lined with these tubes, up to 36 in. in diameter. In one case the depth was 44 ft. Tests were also made to demonstrate the strength of the joint : 12 ft. of tubes were jointed up with cement and

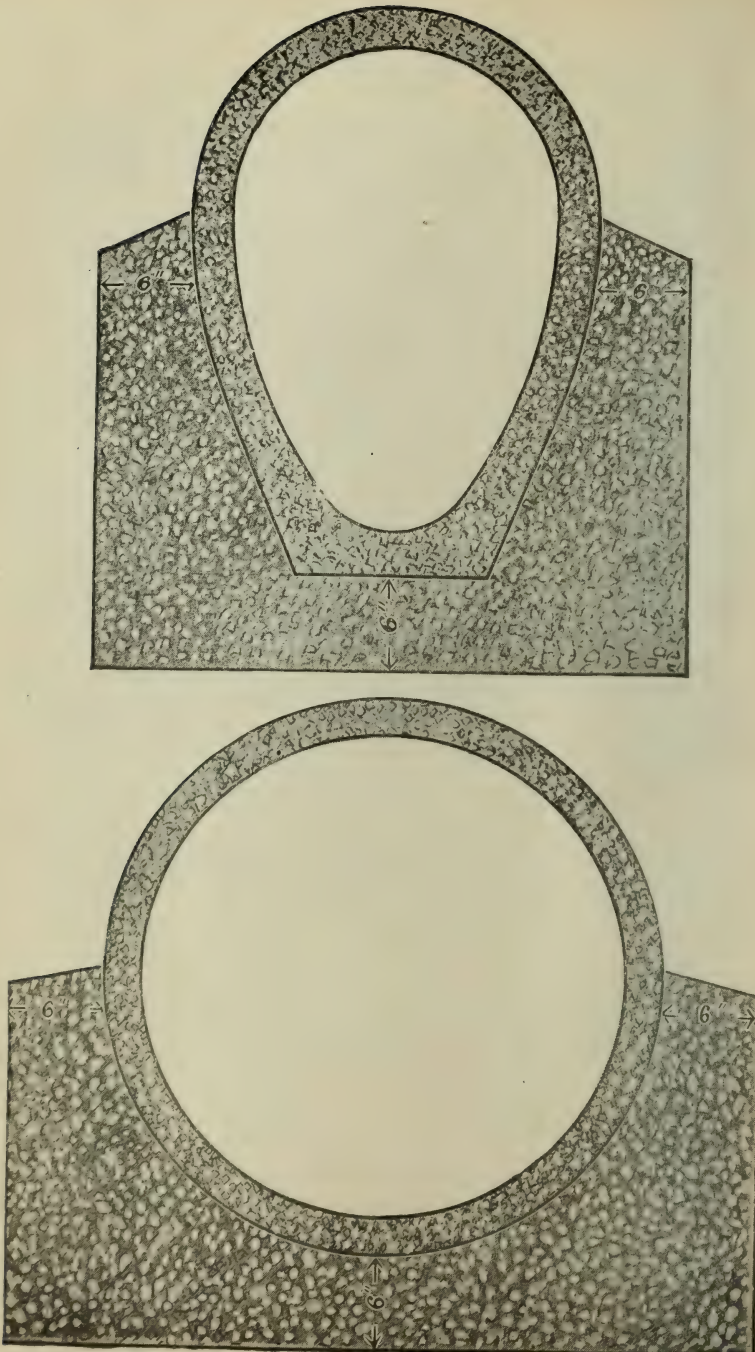


FIG. 1.—Usual Method of laying Concrete Tubes.

supported on bearings 11 ft. apart, weights of nearly 1 ton were placed on the middle tubes, and a fracture took place about 6 in. from the third joint, all the joints remaining sound. A column of tubes was filled with water to 30 ft. in depth without producing any sign of leakage at the joints or percolation through the body of the tubes.

Mr. Butler states that scarcely any English manufacturer's cement is uniformly good, by reason of the lack of care and tests during the various stages of the manufacture. He contrasts this inefficient method with the exact and scientific process followed by two manufacturers, one at Boulogne and one at Stettin, whose cement was reliable and of good quality. He says, "the Stettin cement is of very high quality and guaranteed, after a 7 days' test, to stand 1,280 lb. breaking strain on $2\frac{1}{4}$ sq. in." In another place he says, "Owing to the comparatively thin wall of the pipes, it is a necessity that the cement should be of the very best quality—heavy, slow-setting cement—finely ground to a fifty linear gauge if possible, and with not more than 10 per cent. residuum."

A sample of vitreous stoneware concrete, 35 to 40 years old, similar to that described, is exhibited. The superior strength of such concrete, as compared with gravel concrete, was also shown by the previous series of tests made by Mr. John Grant, M.Inst.C.E., about 1870 (*vide* Vol. 32, pages 296–301, *Minutes of Proceedings of the Institution of Civil Engineers*).

Concrete tubes, unless strongly reinforced, should be laid on a concrete bed not less than 6 in. in thickness, and the concrete be brought up round the tube as far as the springing, as shown in Fig. 1. By the omission of this precaution many failures have occurred. In tubes liable to internal pressure the concrete should entirely surround them.

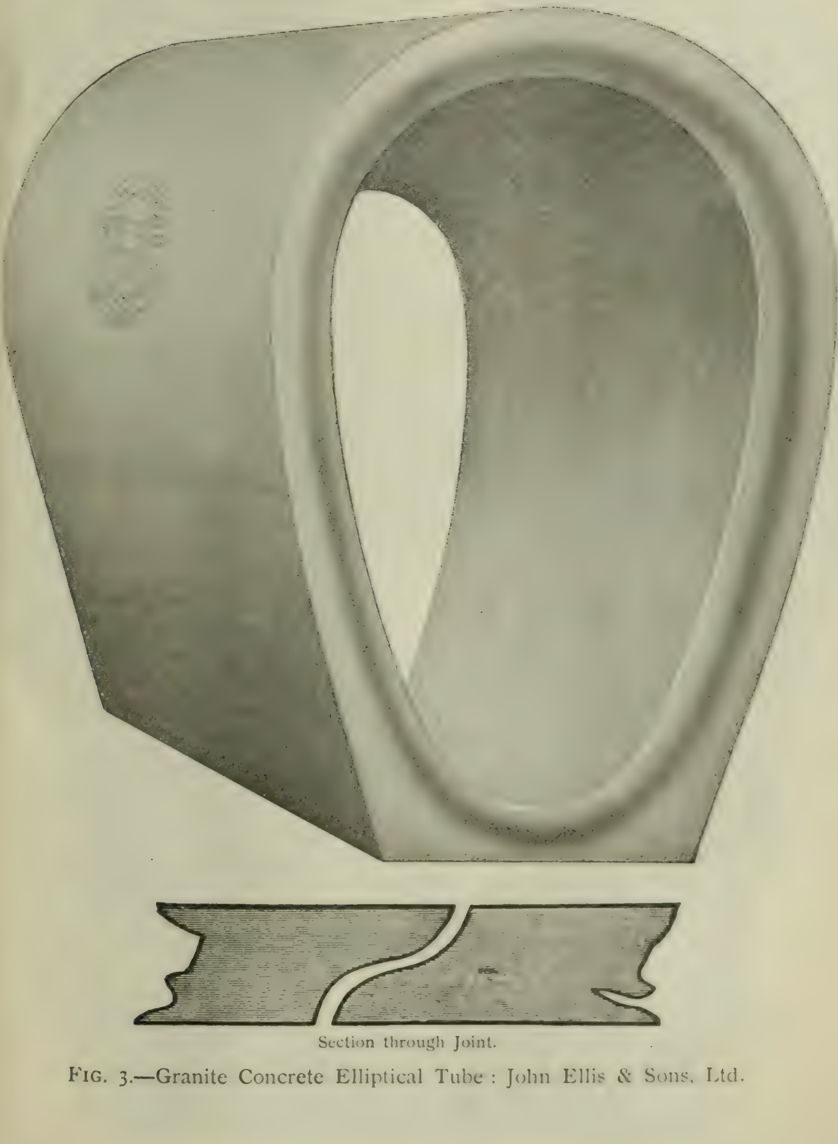
The tubes have ogee, rebated joints, which are luted with cement, and the inside pointed up. In the case of the John Ellis & Sons 60 in. by 40 in. egg-shaped tube, the joint is a square recessed butt (see Figs. 2, 3, 4, and 5).

In a paper read by Mr. E. J. Elford, M.Inst.C.E., on June 6, 1914, before the Institution of Municipal and County Engineers (*vide* Volume XL of *Proceedings*), he describes the method of jointing concrete tubes laid in tunnel, at Southend-on-Sea. The following extract from his paper gives the particulars :—

"The socket and spigot were so designed as to provide when placed together an annular space within the joint of about $\frac{1}{4}$ in., and the specification required the contractor to lay the tubes with dry, clean joints, fill in solidly with

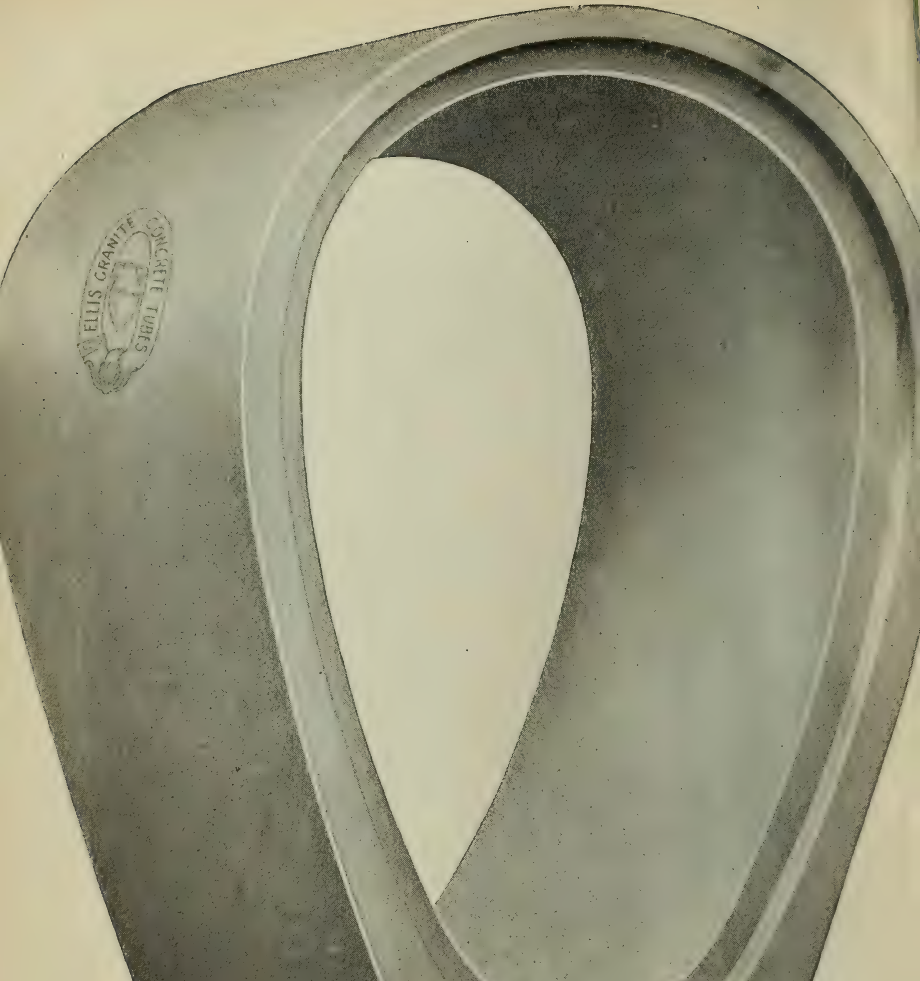
Section through Joint.

FIG. 2.—Granite Concrete Circular Tube : John Ellis & Sons, Ltd.

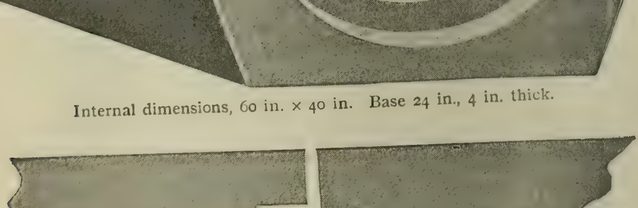


Section through Joint.

FIG. 3.—Granite Concrete Elliptical Tube : John Ellis & Sons, Ltd.



Internal dimensions, 60 in. × 40 in. Base 24 in., 4 in. thick.



Section through Joint.

Fig. 4.—Granite Concrete Elliptical Tube: John Ellis & Sons, Ltd.

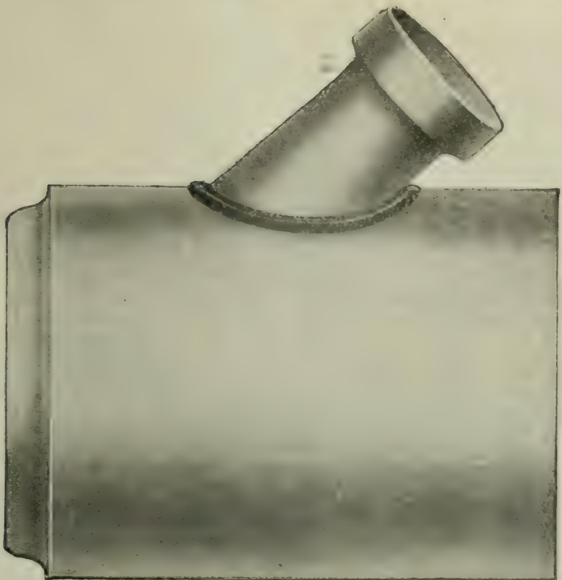


FIG. 5.—Granite Concrete Junction: John Ellis & Sons, Ltd.

FIG. 6.—Granite Concrete Circular Tube: Sharp, Jones & Co.

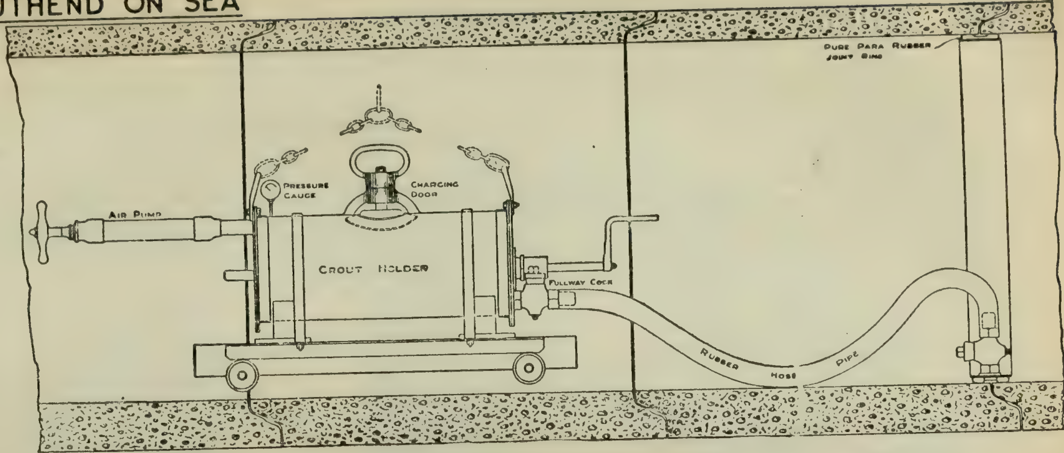
1914

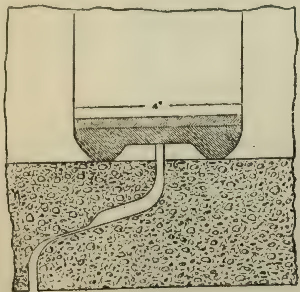
COUNTY BOROUCH OF SOUTHEND ON SEA

MAIN SEWERACE WORKS

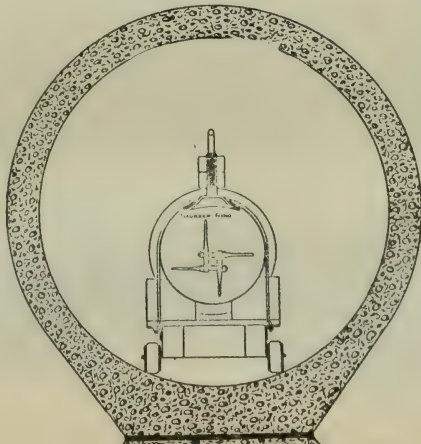
TUBE JOINTINC APPARATUS

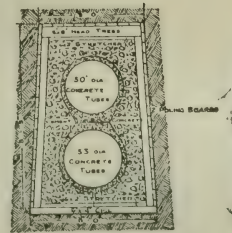
SCALE OF FEET



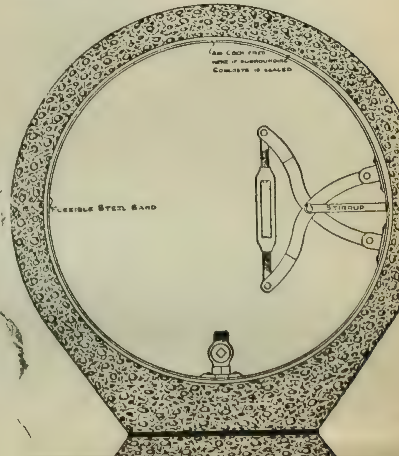


SECTION OF RUBBER RINC AND STEEL BAND BEFORE COMPRESSION





TYPICAL CROSS SECTION OF HEADINC



concrete on the outside, and afterwards to make the joints by first sealing the interior with an expanding steel and rubber ring, and then forcing liquid cement into the annular space through a hole in the ring at the side or invert. A second hole was to be provided at the top of the ring as an air vent, and this was to be left open until the cement began to flow out, after which it was to be plugged and the cement forced in until the joint was solid."

The joints were quite satisfactory. Fig. 7 shows the arrangement for pipe jointing.

Concrete tubes are manufactured on the "Jagger" system without ramming the matrix. The moulds are placed on a

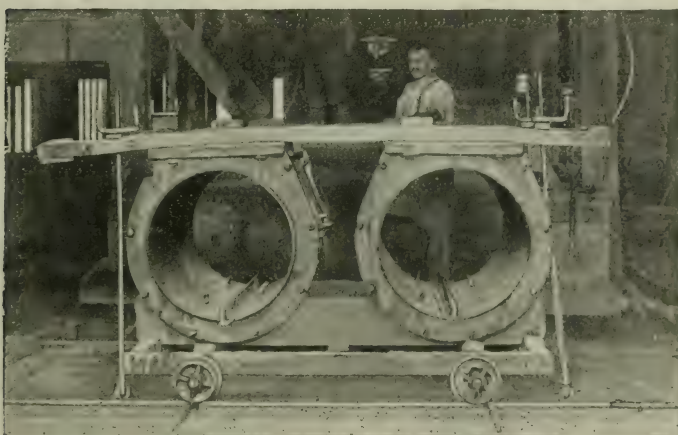


FIG. 8.—"Jagger" Vibrating Table, with Two Moulds for Concrete Tubes.

table to which a motion is imparted, described as "a horizontal vibratory movement combined with a vertical oscillation accompanied by a suddenly arrested rocking movement which might be expressed as intermittent gliding shocks in quick rotation." The mixture is fed into the moulds, and the effect of the movements of the table is to cause the aggregates to settle down into a compact mass, without stratification, before the initial set of the cement occurs. It is also claimed that all voids disappear. The presence of reinforcement does not interfere with the solidification of the concrete.

Fig. 8 shows the moulds on the table. It will be noticed

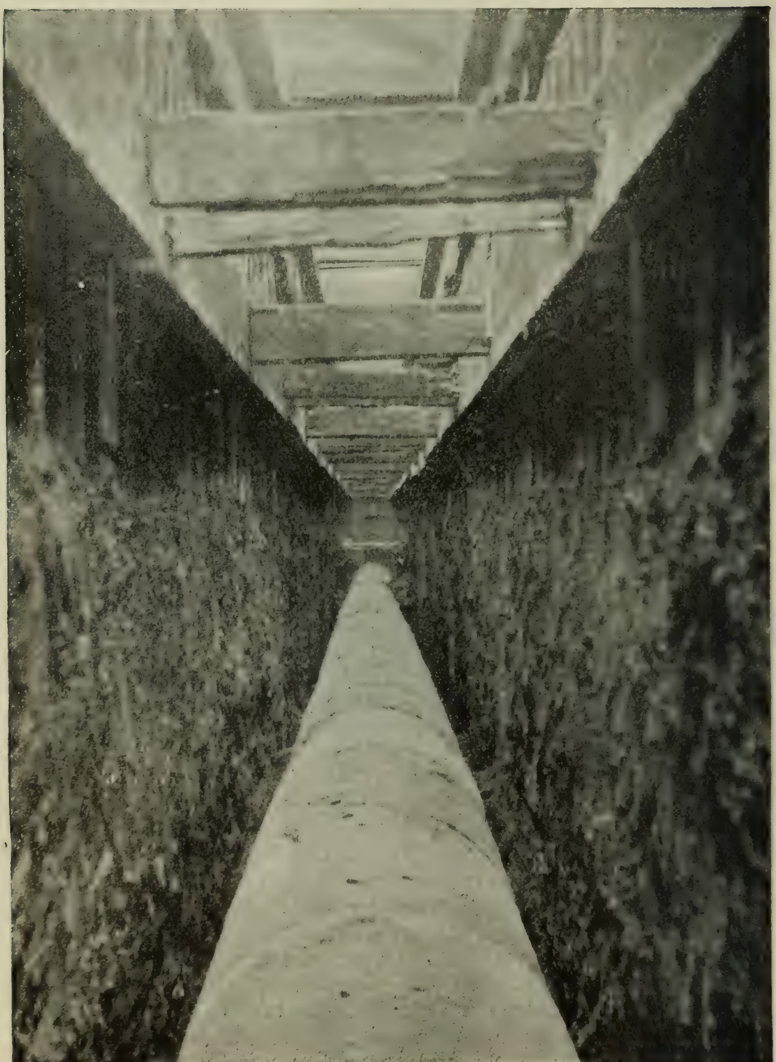


FIG. 9.—Granite Concrete Tubes laid in Trench.

the tubes are cast crown downwards ; the action of the table brings the "slimes" in the cement to the surface, where they can be removed by the attendant.

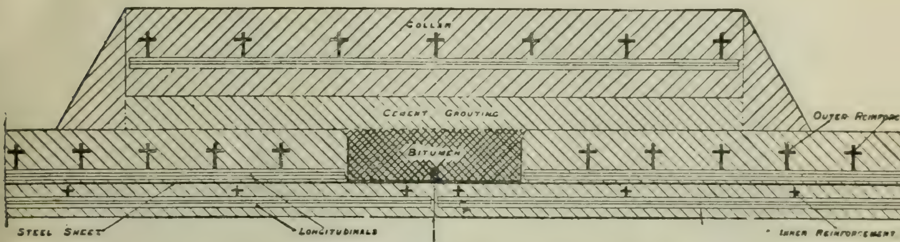


FIG. 10.—"Bonna" Pipe : Section through Pipe and Collar.

A model pipe made on this system is exhibited ; it shows a smooth, hard surface.

In bad ground it is advisable to strengthen the concrete

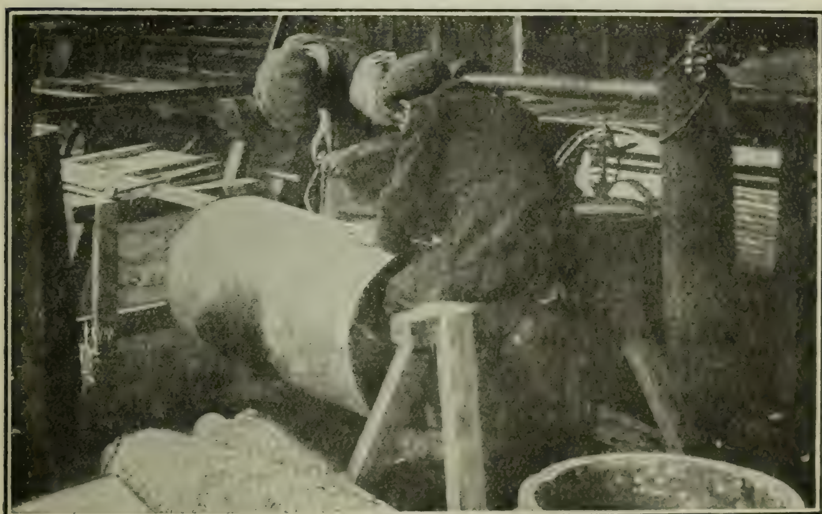


FIG. 11.—"Bonna" Pipe : Welding Steel Sheets.

under the tubes by inserting reinforcement therein, thus forming a raft foundation.

In some instances artificial means are adopted to hasten

the setting of the concrete while the tube is in the mould by exposure to hot air, with a view of releasing the moulds more quickly. It is claimed that the precautions taken during this process prevent any weakening of the resisting power of the concrete. It would appear that this is a case where investigation is required to ascertain more definite information.

For sewers, the tubes are usually 3 ft. in length, and are sometimes slightly reinforced by the insertion of three rings of steel ¼ in. diameter, spaced about 12 in. apart. The

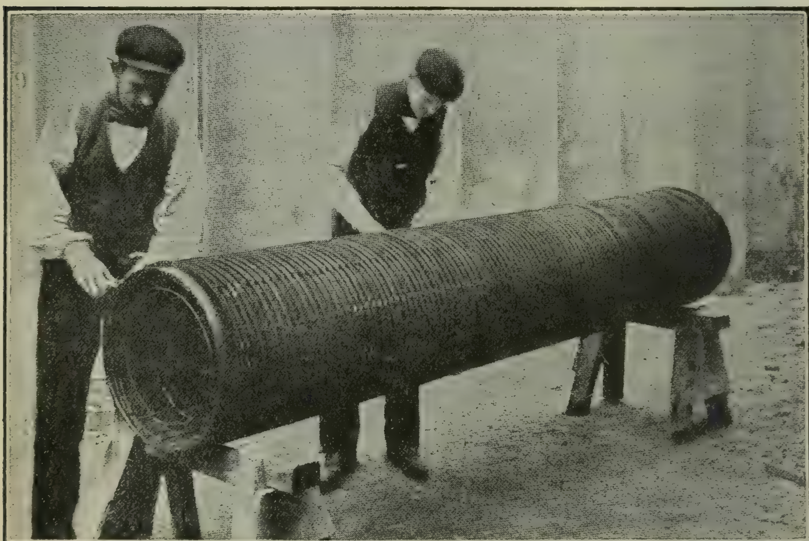


FIG. 12.—"Bonna" Pipe: Application of External Reinforcement.

joints in the rings are welded. There are no longitudinal nor spiral rods. It is found that this simple type of reinforcement gives to the tube increased resistance to crushing.

Tests were made in the author's presence with three tubes of plain granite concrete and three tubes of granite concrete with the three rings of ¼ in. steel. The tubes were about twelve months old, 3 ft. in length, 27 in. in diameter, and were, for the test, bedded in sand half-way up. The load was distributed along the crown of the arch. The reinforced tubes cracked along the invert with an average load of 7½ tons, but continued to support the load. The plain

concrete tubes failed completely at an average load of $4\frac{1}{2}$ tons. The thickness of concrete was 2 in., and the mixture the same in each case.

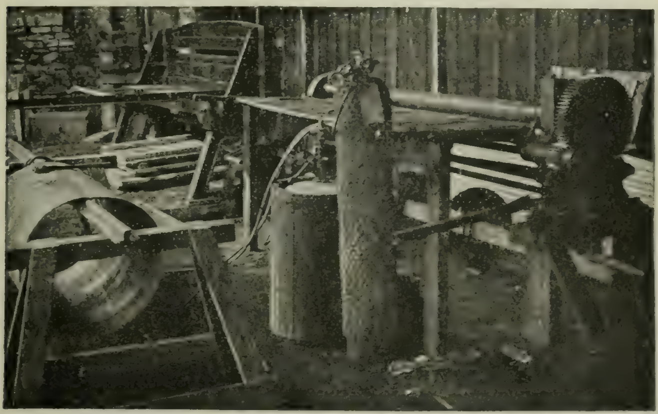


Fig. 13.—"Bonna" Pipe: Appliances for Handling Edges of Steel Sheets.

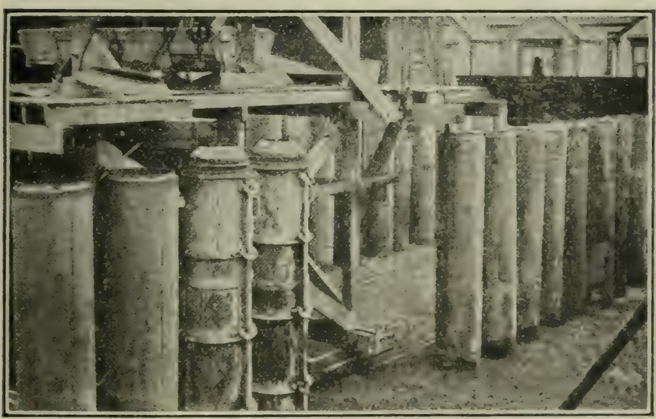


Fig. 14.—"Bonna" Pipe: Casting.

Tubes of the above type are very suitable for sewers, but are not adapted to withstand high internal pressure nor shock from moving columns of water. Fig. 9 shows a line of tubes laid in the trench, before filling in.

Tubes are made capable of withstanding high internal pressure, and of resisting water hammer. They are constructed with various systems of reinforcement, and are in

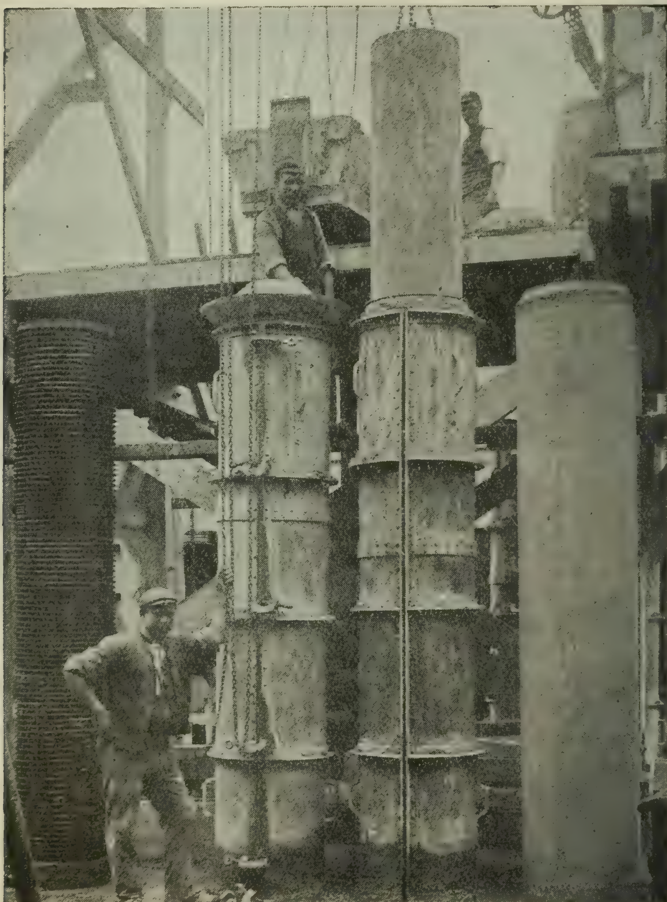


FIG. 15.—"Bonna" Pipe: Cores being withdrawn.

use in Great Britain and more generally abroad. It is not possible to refer to all the systems of reinforcement adopted. Generally they consist of various combinations of helicals and longitudinals, with the addition in some instances of circulars.

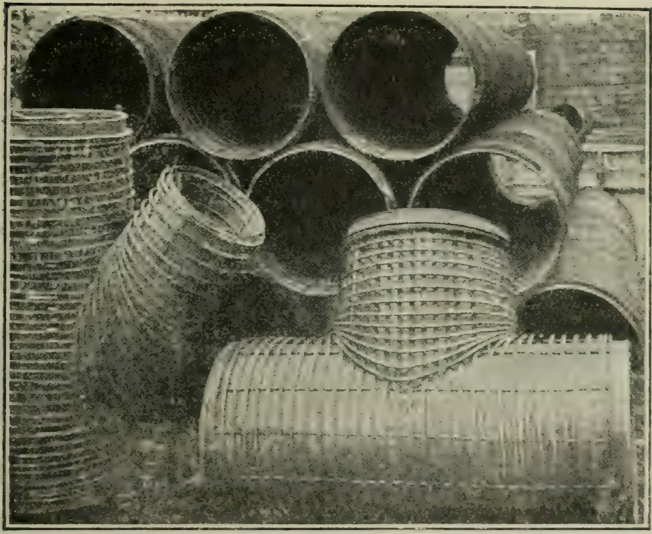


FIG. 16.—"Bonna" Pipe: Reinforcement of Junctions.

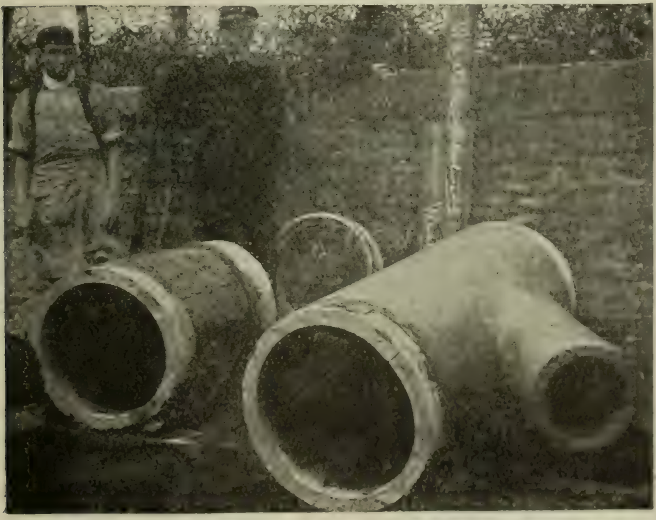


FIG. 17.—"Bonna" Pipe: Hand Moulding of Junctions.

The "Bonna" system goes a step farther, and is essentially a thin steel pipe strengthened and protected, externally and internally, by reinforced concrete. The internal reinforcement is not taken into consideration in the stress calculations, but its presence is necessary to support the internal concrete. The dimensions of the cruciform steel bar forming the helicals, and the distances of the helicals from one another, together with the thickness of the concrete, vary according to the diameter of the pipe and the pressure to be resisted.

The pipes are frequently manufactured on the site of the works where they are to be laid. They are usually 10 ft. in

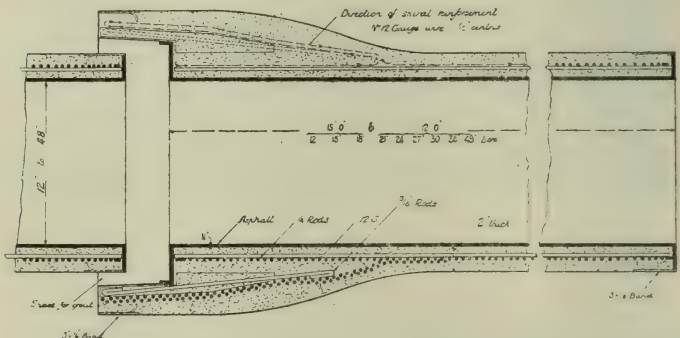


FIG. 18.—Section of "Siegwart" Socket and Spigot Pipe.

length, and have been made from 10 to 78 in. in diameter. In one instance at Rio Soza, in Spain, two syphons of Bonna pipes, each 12 ft. 6 in. diameter, were laid under two canals.

Bonna pipes have been used for water mains at Swansea against heads of 246 ft. and 500 ft., at Norwich for sewage rising main, head 131 ft., and at Clydebank for water head 390 ft., and are about to be laid at Leeds for water. They have been largely used on the Continent, and 119 miles of 10-in. to 49-in. water mains were in course of construction in Flanders at the outbreak of the war. They have also been largely used in Paris. Twenty years since, the Municipality laid down one mile of 6 ft. diameter as a rising main for sewage, head 115 ft., in a gallery alongside a steel riveted main of same diameter. It has been found that the steel main requires considerable maintenance, and the concrete tube none. Further particulars of this work, with illustrations, are given in an account of a visit to Paris in 1910 of a deputation from the Concrete Institute published in the

TRANSACTIONS, Vol. II, Part II, p. xl. In connection therewith is described and illustrated a Bonna pipe which was removed after being in service for thirteen years. When cut into in the presence of the deputation and examined after sixteen years it was found that the steel was practically clean and

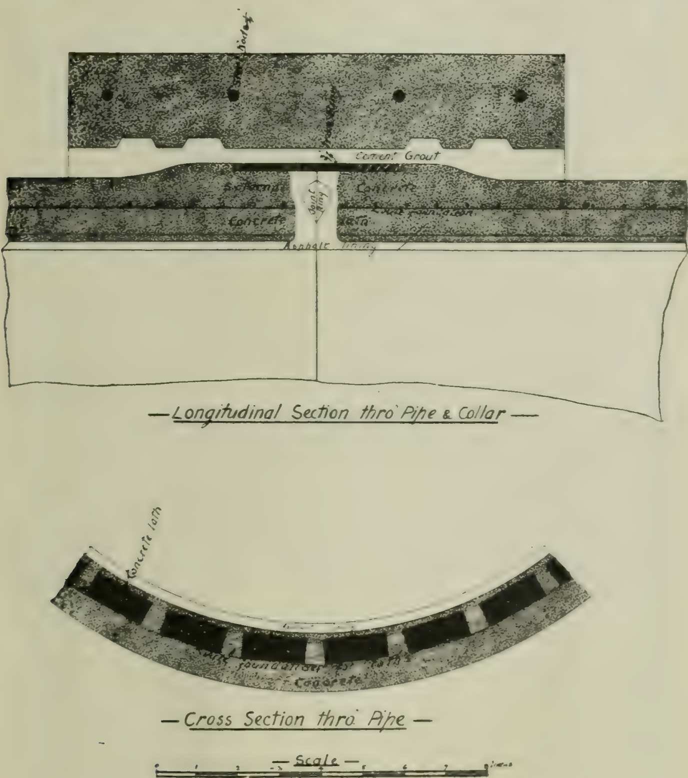


FIG. 19.—Section of "Siegwart" Butt-ended Pipes with Collar.

good, and that the concrete was lying close against it. There were slight traces of rust on parts of the inner reinforcement but the rusting action seemed to be dormant although the protective covering of concrete was only about $\frac{1}{4}$ in. Portions of this pipe were sent to the Institute shortly after the visit and are now on exhibition at this meeting (see Figs. 10, 11, 12, 13, 14, 15, 16, and 17).

The "Siegwart" reinforced pipes are constructed on a most ingenious system. A steel core is revolved whilst concrete is applied by machinery until the required thick-

FIG. 20.—"Siegwart" Socket and Spigot Pipes laid in Trench.

ness is obtained. Steel wire is then wound by the same machine spirally over the concrete, in one or more layers as required. The assembled longitudinal rods are slipped

over the spirals, and further concrete applied by the machine until the pipe is finished.

Siegwart pipes are made 15 ft. long for diameters 12 in.

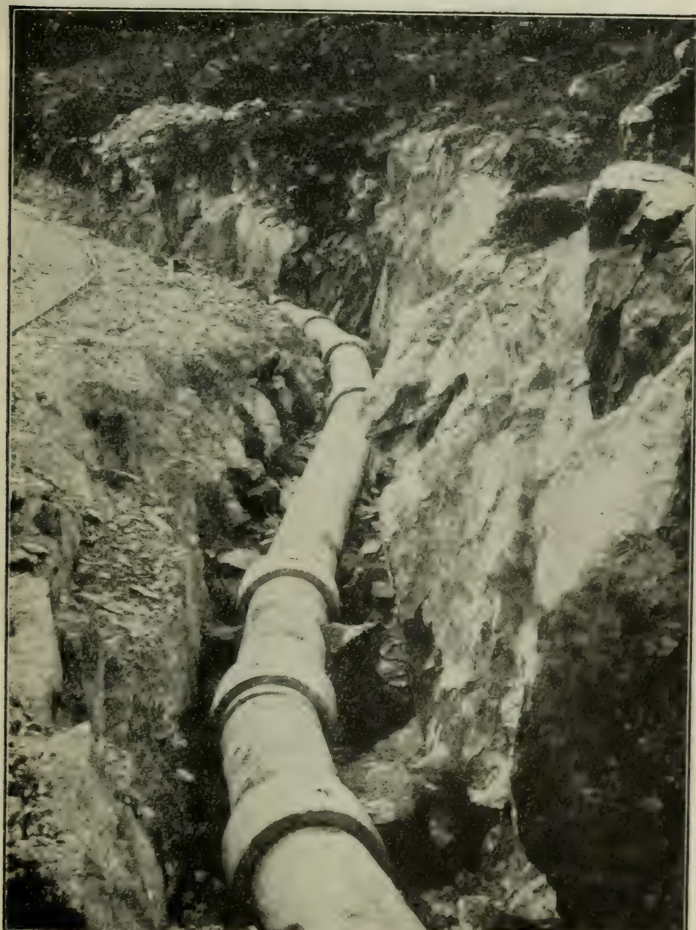


Fig. 21.—"Siegwart" Collar Pipes laid in Trench.

to 18 in., and 12 ft. long for larger diameters. They have been used for water mains in Cairo, Zurich, Italy, and as a pumping main at Grays, Essex, the pressures being from

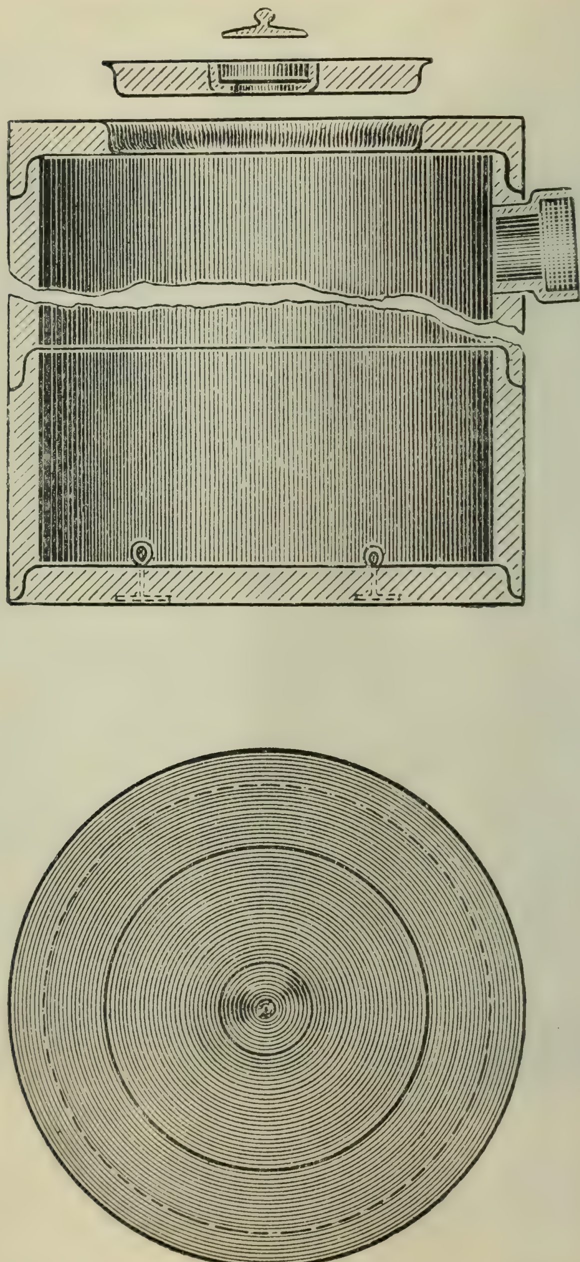


FIG. 23.—Concrete Tubes applied to Cesspits : Sharp, Jones & Co.

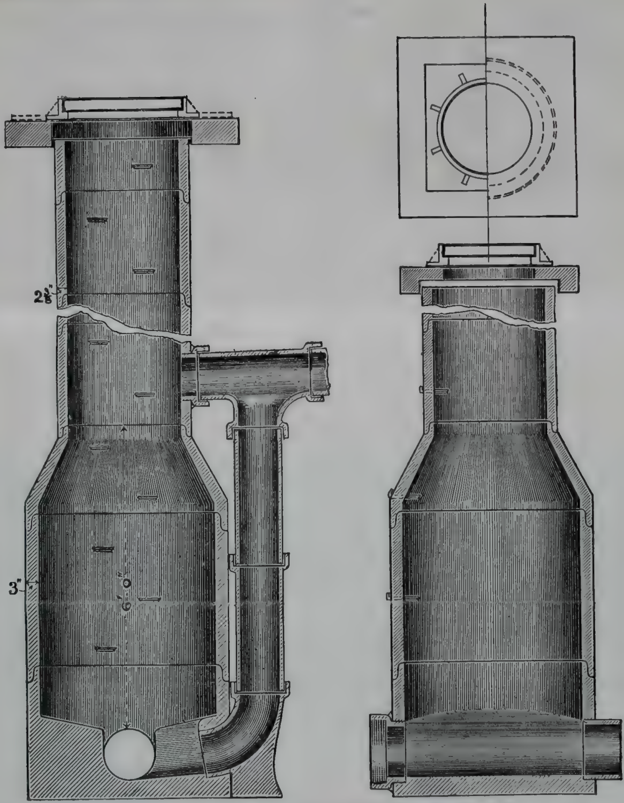


FIG. 22.—Manholes in Concrete Tubes, with Cascade Branch Sewer and Special Stoneware Junction Block and Foot-bend : Sharp, Jones & Co.

20 to 100 lb. per sq. in., and have been tested at the Grays works to 300 lb. per sq. in.

Both socket and spigot pipes, and butt-end pipes, with reinforced collars are made. Fig. 18 shows the former and Fig. 19 the latter. Fig. 20 shows the collar pipes and Fig. 21 the socket and spigot pipes as laid.

An Appendix will be found at the end of this paper, giving a description of the manufacture of concrete tubes furnished respectively by the following firms : Messrs. John Ellis & Sons, Messrs. Sharp, Jones & Co., Messrs. Hughes & Lancaster (Bonna System), and Messrs. Siegwart, Ltd.

Concrete tubes used as sewers may suffer damage from liquids being discharged into them at high temperatures, also by the discharge of acids. Instances have been recorded

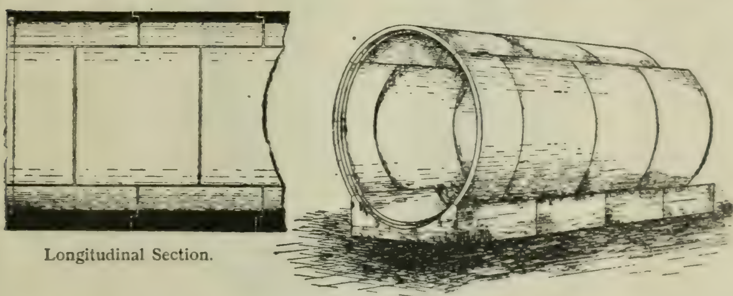


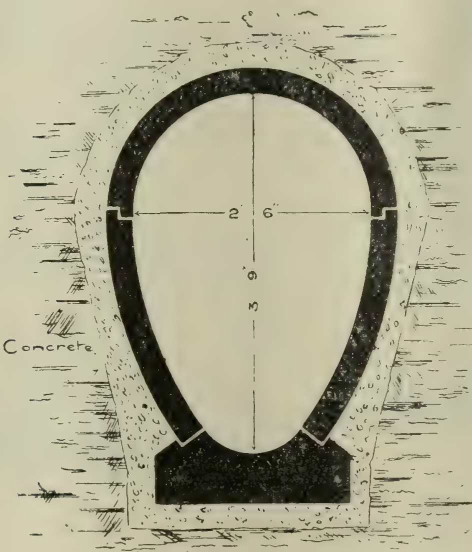
FIG. 24.—A 54-in. Circular Concrete Sewer : Dean Stone Company.

where concrete has perished when in contact with sewage. A paper was read by Mr. Sidney H. Chambers before this Institute in April 1910 describing the disintegratory changes in the concrete of the sewage tanks and conduits that occurred at Hampton. To ascertain if concrete tubes would be affected he caused a couple of 9-in. tubes to be placed in the tank for a period of eight months. One was coated with Dr. Angus Smith's composition. The coated tube was not affected. The uncoated tube was attacked over the area that was immersed in the sewage at high level, but which was not in contact therewith at low level. He does not state whether the uncoated tube had been, on removal from the moulds, dipped in a bath of solution of silicate of soda.

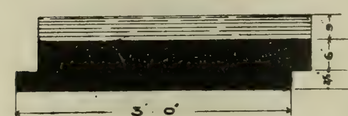
In the great majority of cases concrete suffers no damage by contact with sewage. Still, under certain conditions the lime in the cement may be attacked, and, as far as is possible,

precautions should be taken to prevent such conditions arising. Further investigation is needed to ascertain why concrete occasionally disintegrates when in contact with sewage.

Concrete tubes are constructed to form manholes, water-tanks, and cesspits.



Transverse Section.

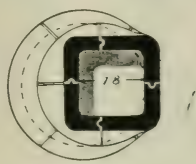


Longitudinal Section of Invert.

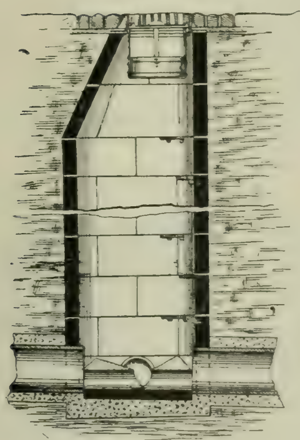
FIG. 25.—Elliptical Concrete Sewer: Dean Stone Company.

Fig. 22 shows the method adopted by Messrs. Sharp, Jones & Co. for manholes. The foundation block is of concrete, made in one piece with the inverts moulded, for the main sewer and the branches joining the same. It is lowered into the trench by the aid of ring bolts, then the tubes forming the chamber are added, and as many feet of

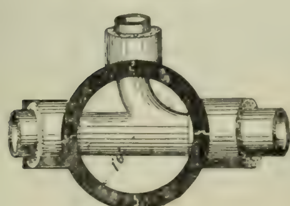
tubes forming the shaft as are required. Foot irons are provided. The shaft is covered with a special block of concrete, which carries the iron cover and frame.



Plan at Top.



Vertical Section.



Plan

FIG. 26.—A Concrete Manhole : Dean Stone Company.

Fig. 23 shows the application of concrete tubes to cesspits. Sewers, reservoirs, manholes, etc., are constructed with moulded blocks of concrete which are supplied by various manufacturers.

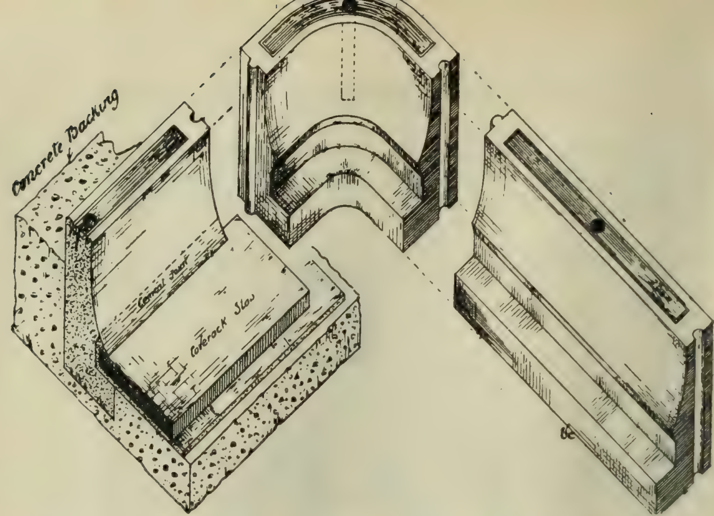


FIG. 27.—Angle Block and General Method of Construction of Concrete Reservoir: Dean Stone Company.

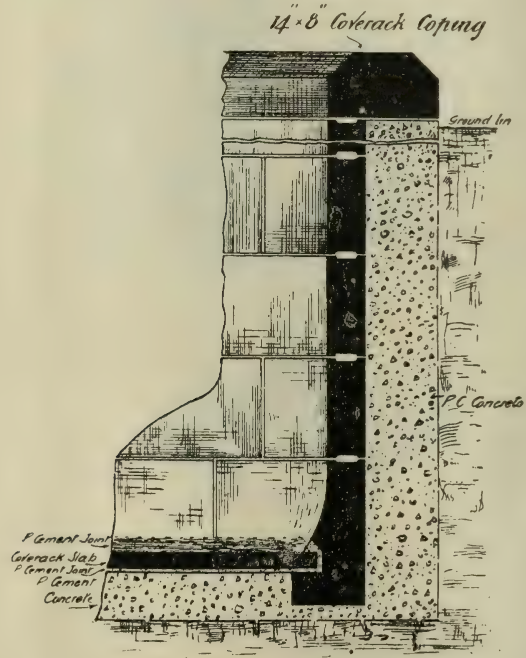


FIG. 28.—Vertical Section of Wall showing General Method of Construction of Concrete Reservoir: Dean Stone Company.

The Dean Stone Company, of Devonport, manufacture blocks from crushed syenite obtained from the company's quarries at St. Keverne, Cornwall, and Portland cement. No machinery is used for mixing nor for pressing the mixture into the moulds. The blocks are moulded for circular and for egg-shaped sewers. Figs. 24 and 25 show these sewers. It will be noticed that the blocks break joint. Segmental blocks are also made for manholes as shown in Fig. 26.

Straight and angle blocks are made for reservoir construction as shown by Figs. 27 and 28, whilst Fig. 29 is a

FIG. 29.—Concrete Reservoir: Dean Stone Company.

photograph of a reservoir built of these blocks. The blocks are $4\frac{1}{2}$ in. thick and have $1\frac{1}{2}$ in. hole and frog. They are 1 ft. in height, and of varying lengths to admit of breaking joint at each course. The bottom course is made stronger, the blocks radiating from $4\frac{1}{2}$ in. thick at the top to 9 in. at the bottom, with a special 4 in. base, with groove to pin in the floor paving blocks. Liquid grout, composed of 1 part cement to 1 part of fine-sifted crushed syenite, is, when the wall coursing is in position, poured down the $1\frac{1}{2}$ in. hole, which it fills together with the frog underneath.

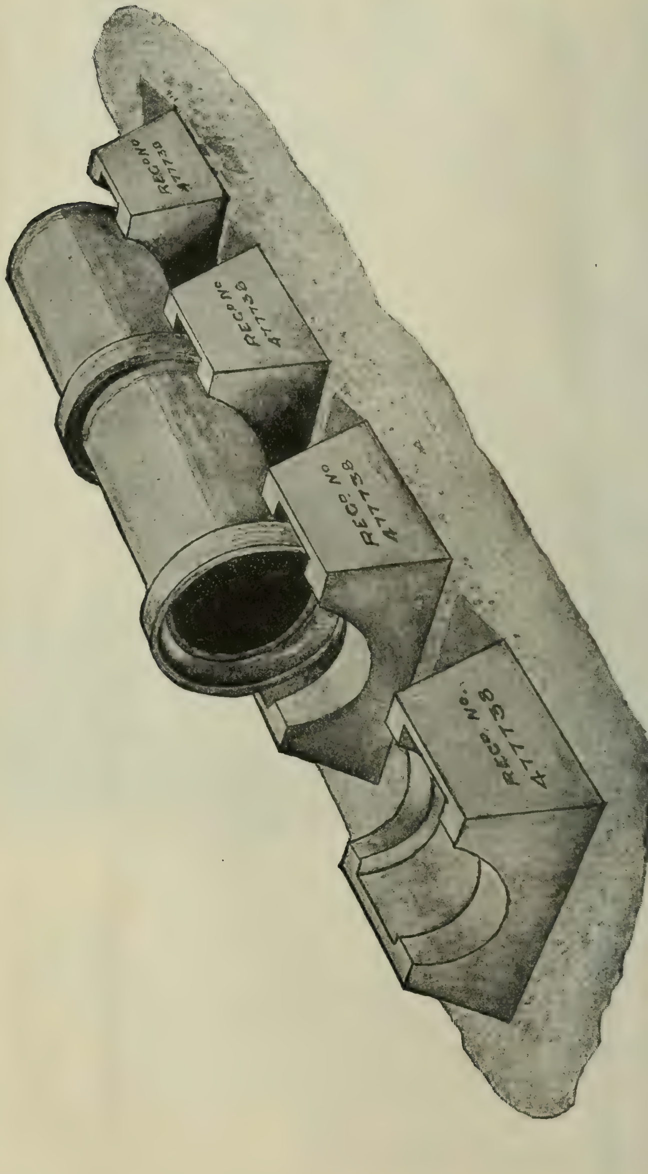


FIG. 30.—Foundation Blocks for Sewer Pipes : Dean Stone Company.

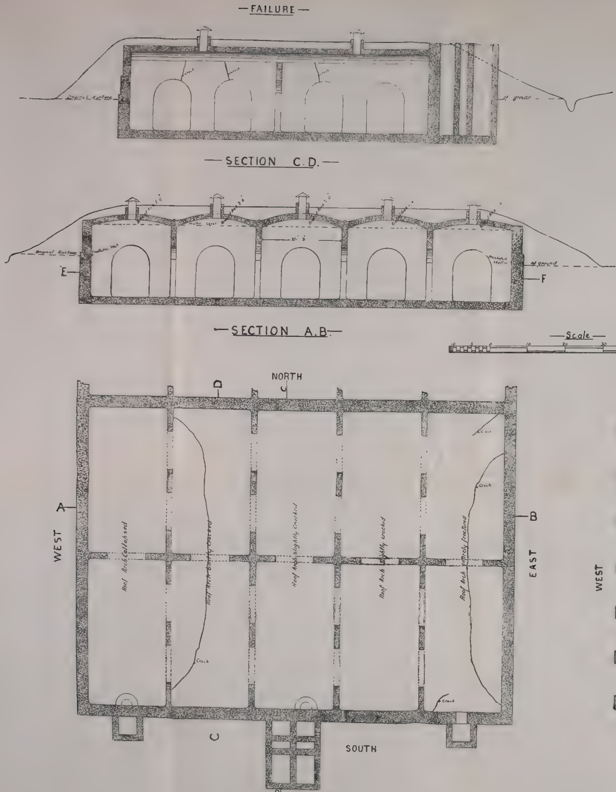


Fig. 31

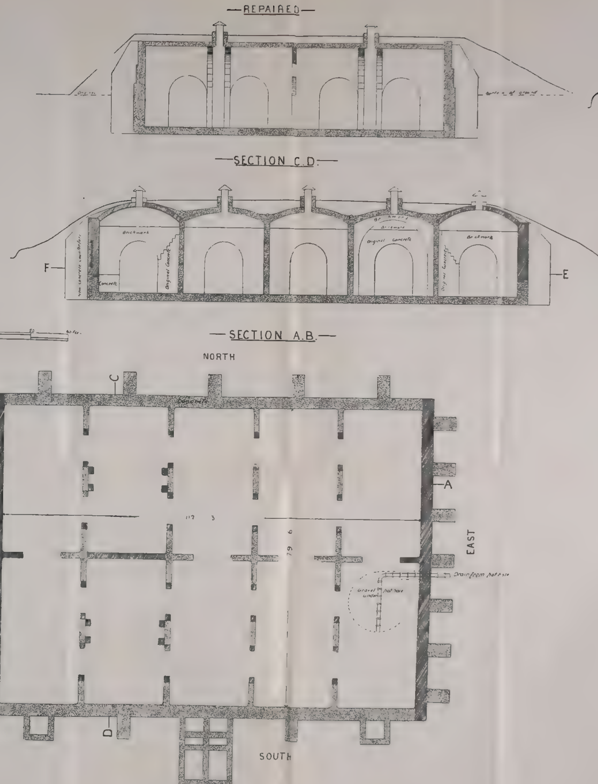


Fig. 32.

Concrete Reservoir. Failure and Reconstruction.

Concrete blocks are also made by the Dean Stone Company, as shown by Fig. 30, to provide a bearing under each joint of an ordinary stoneware pipe sewer. When the pipes are in position, liquid cement is poured into the opening left between the outside of the pipe collar and the block, until the cement stands nearly level with the upper surface of the block ; by this means a good bottom joint is made in the sewer.

The following instances within the author's experience are given as illustrating, in greater detail, the application of concrete to sanitation works :—

Failure and reconstruction of concrete-covered reservoir.
Temperature cracks in concrete reservoir wall.
Ejector chamber in concrete with steel-plate lining.
Concrete mole forming sewage outfall into sea.
Circular hydrolytic tanks in reinforced concrete.

Failure and Reconstruction of Concrete-covered Reservoir.

This reservoir was constructed in concrete in the year 1889, and stood for two years when the failure occurred. Fig. 31 is a plan and section of reservoir as originally constructed ; Fig. 32 plan and section of reservoir as reconstructed. Figs. 33 and 34 are two photographs of the fallen roof arch in Bay No. 1.

The internal dimensions were :—

East to west	112 ft. 3 in.
North to south	79 ft. 6 in.
Capacity	one million gallons
Level of invert of overflow ...	18 ft. 10 in. above floor

The reservoir was divided into two parts by a longitudinal central division wall from east to west, and into five bays by four transverse walls from north to south ; all the interior walls had arched openings. The bays were roofed in by concrete arches springing from the east and west walls and the transverse walls.

Ample ventilation was provided by ten ventilators in the roof, each 18 in. in diameter. The roof was covered with soil to a depth of 2 ft. over the crown of the central bay, and 1 ft. 6 in. over the outside bays.

The reservoir was partly in and partly out of ground, the floor line being about 11 ft. below ground level. The

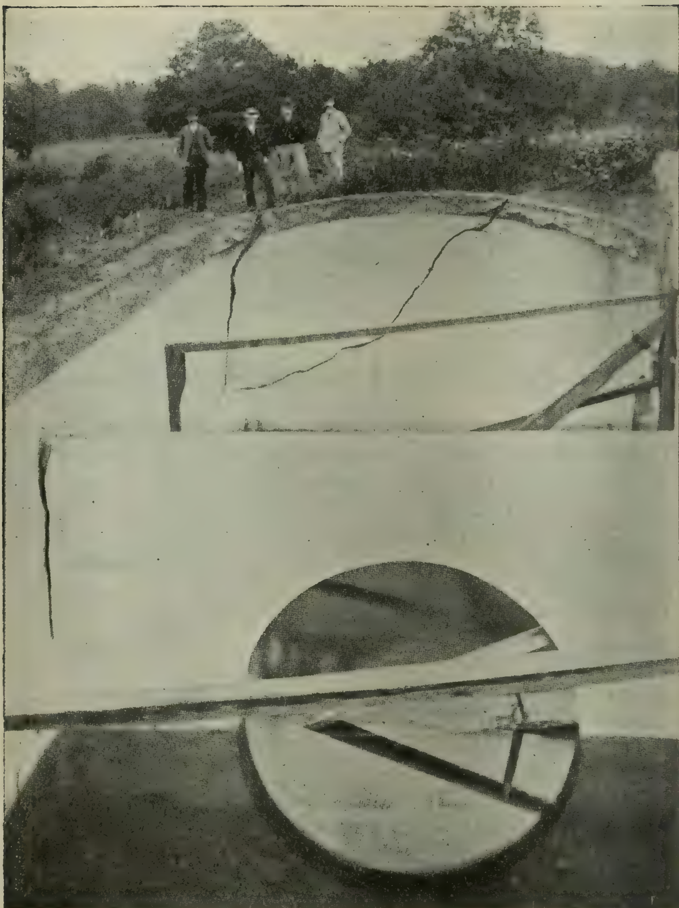


FIG. 33.—Concrete Reservoir Failure: West Bay, looking North.

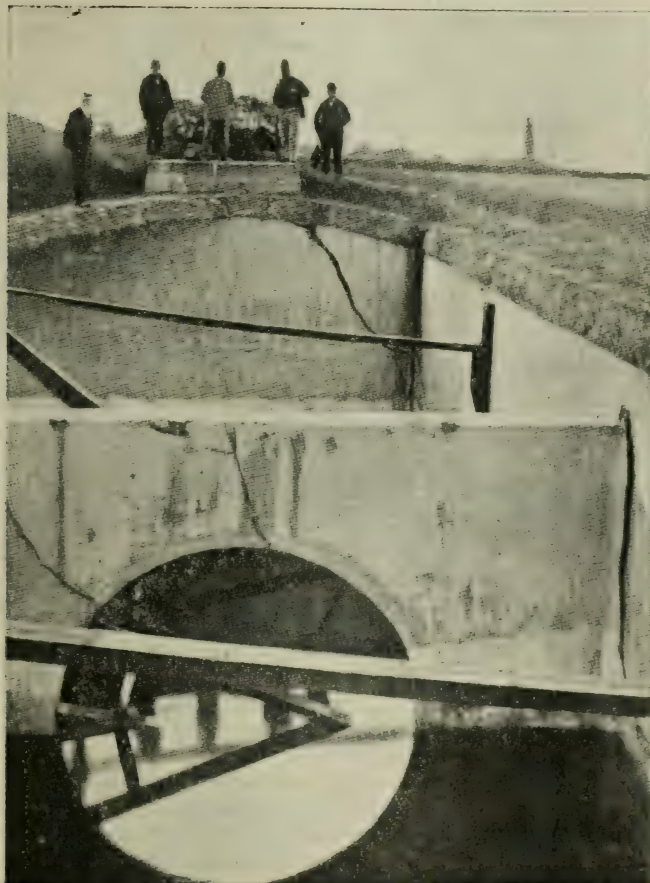


FIG. 34.—Concrete Reservoir Failure : West Bay, looking South.

subsoil was clay, with a pot-hole of gravel in the east bay, in which was a weak spring of water. Earth banks, with slopes varying from $1\frac{1}{2}$ to 1, to 1 to 1, were placed round the walls where out of ground.

The thicknesses of the exterior walls were :—

East and west walls which received the thrust of the arch	3 ft. for a distance of 9 ft. 6 in. from the floor
,, ,, ,,	2 ft. 6 in. for a further 10 ft. 9 in., being up to the top of the skew-backs
North and south walls ...	3 ft. for 9 ft. 6 in. from floor
,, ,, ,,	2 ft. 6 in. for a further 5 ft. 6 in.
,, ,, ,,	2 ft. for a further 5 ft. 3 in. and carried up beyond to the soffit of the arch

The walls were without counterforts. The south wall was strengthened by the addition of three valve chambers.

The floor was 2 ft. in thickness throughout.

The clear span of the roof arches was 21 ft. 3 in., and the rises given to the segmental arches were :—

Centre arch ...	3 ft.	about	0·14	of the	span
Next arches ...	2 ft. 6 in.	,,	0·118	,,	,,
Outer arches ...	1 ft. 9 in.	,,	0·08	,,	,,

The thickness of concrete in the arches was 1 ft. at the crown and 1 ft. 3 in. at 2 ft. from the springing.

Water was pumped into the reservoir through a rising main 10 in. in diameter.

The pumping station was situated in a valley about half a mile from and out of sight of the reservoir. No apparatus was fixed in the pump-house for indicating the level of the water in the reservoir, beyond a pressure gauge on the rising main, which main entered the reservoir just above the floor level.

The overflow pipe was 10 in. in diameter, and throughout its length no curved bends were used, but only direct right angles. To run full bore necessitated the water standing in the reservoir to at least a height of 19 ft. 8 in. above the floor level. After the failure it was ascertained that the level of the water had reached 20 ft. 8 in., being 1 ft. 8 in. above the springing of the arches.

The concrete used in the work was composed of Thames ballast and Portland cement specified 6 to 1 mixture. It was, on examination after the failure, found to be dense, hard, and well set, except in the part of the floor over the pot-hole, and great difficulty was experienced when the repairs were effected in cutting the benchings in the undamaged portions of the walls to receive the new work. The interior surfaces of the exterior walls were rendered in cement.

For some days prior to, and on the day of the failure, continuous pumping into the reservoir had been in operation.

The reservoir collapsed suddenly without any previous signs of weakness having been detected, the west wall breaking away from the arch, the north and south walls, and the central interior wall.

The west wall was fractured horizontally throughout its length, the line of fracture being practically along the set-off from 3 ft. to 2 ft. 6 in. in the thickness. The fractured portion remained in position, but was pushed over until the top was 4 in. out of plumb.

The east wall was badly cracked for the entire length along a line about 5 ft. above floor level, being about 4 ft. 6 in. below the set-off. Fractures occurred at the junctures with the north and south walls and the interior division wall. At a point in the wall nearest to the gravel pot-hole under the floor, the east wall was badly cracked vertically from top to bottom.

The north and south walls were only affected at the junctures with the east and west walls, where triangular pieces were broken off from their upper portions.

The four interior transverse walls were badly strained, the arches over the openings being cracked.

The roof arch over the western bay collapsed completely, falling into the reservoir. The next arch was cracked throughout its length. The arches over the other two interior bays were but slightly affected. The eastern arch was fractured for its entire length, but remained in position.

The floor over the pot-hole was humped up about 4 in. above floor level in the centre of a circle about 16 ft. in diameter. On examination it was found that other causes than the collapse of the reservoir had effected this result. The rest of the floor was in good condition.

The failure was due to faulty design, coupled possibly with over-pumping. The east and west walls were not strong enough to withstand the stresses due to the thrust of the arches and nearly 21 ft. head of water. The arches had

not sufficient rise, especially those of the east and west bays, and if they were of sufficient strength at the crowns, they should have been strengthened at the haunches. Structurally the north wall was the weakest, but in this case there was no arch thrust, the arch acting as a tie, and thereby taking up some of the strain.

Below the horizontal fractures the east and west walls were sound, except for the vertical crack in the former.

Part of the concrete in the floor was found to be covered with a layer, 3 in. thick, of cement and fine furnace slag, or some kindred material. When wet, this layer gave off a strong smell of sulphuretted hydrogen. The layer was penetrated by numerous pinholes and was porous. A piece weighing $6\frac{1}{2}$ lb. dry, took up 1 lb. of water in 24 hours immersion. The concrete under the layer at the pot-hole was affected, and in process of disintegration. When the reservoir was empty, subsoil water oozed through from the pot-hole. It was surmised that sulphur from the slag had attacked the lime in the cement, and caused disintegration in this portion of the concrete.

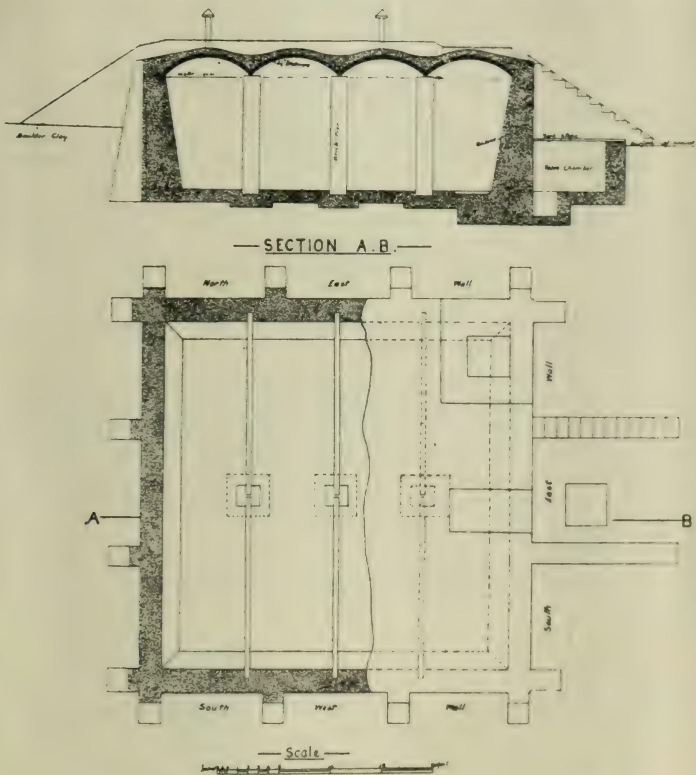
The repairs consisted of reconstructing the east and west walls in brickwork and cement stepped into the undamaged lower portions of the original concrete, with the addition of concrete counterforts extending the full depth of the walls. Counterforts were also put into the north and south walls. The interior walls, where fractured, were reconstructed in brickwork, and brickwork rings, supported on brick piers, turned under the arches over the openings. The east and west roof arches were reconstructed in brickwork and given 4 ft. rise. The adjoining arch to the west arch was strengthened by five brick arches turned under it. The concrete over the pot-hole was taken out and fresh concrete put in, the spring being drained clear of the site. The slag ayer was removed. The overflow was reconstructed with easy bends, and an electric tell-tale connected in the pump-house.

Since reconstruction the reservoir has been, and now is, in continuous use, without needing repair.

Temperature Cracks in Concrete Reservoir Wall, Hadham. (Fig. 35.)

This reservoir was constructed in the year 1905. Depth of water, 11 ft. Subsoil, boulder clay. The floor of the reservoir was 6 ft. below ground level, and the roof and sides were covered by earth, except for a length of 10 ft. on the

south-east wall, where the outer face of the wall was exposed to allow of access to the valve chamber, which was sunk into the ground and extended outwards from the lower portion of the reservoir wall for a distance of 7 ft. The chamber was covered with York stone about 6 in. above ground level,



[FIG. 35.—Hadham Reservoir : Temperature Cracks.

therefore for this length of 10 ft. the reservoir wall, for 8 ft. deep measured from the top, was exposed to the atmosphere, and for the bottom 5 ft. was protected somewhat by the stone covering to the chamber.

The reservoir was entirely in concrete, except that brick piers were used to carry the rolled joists supporting the roof arches, and these arches had a $4\frac{1}{2}$ in. ring of brickwork under the concrete.

The concrete was 6 to 1 by measure of Thames ballast and Portland cement. The materials were turned twice dry and three times wet. The walls were 3 ft. 6 in. thick at the bottom and 1 ft. 8 in. at the springing of the roof arches.

The interior faces of the walls were rendered.

In the latter part of August 1911 it was reported cracks had developed in the south-east wall both inside and outside the reservoir. On examination it was found that the rendering on the inside face was cracked in several places, along its entire length, and that small cracks had formed in the concrete on the outer face where exposed. Also the portions of the rendering on the north-east and south-west walls adjoining the south-east wall were cracked. On cutting out the cracks on the outer exposed face they were found to penetrate into the concrete about 1½ in., and those inside about 2 in. No signs of settlement could be detected by levelling.

In October the cracks were cut out and made good, and have not since reappeared.

July and August 1911 were very hot months, with long periods of sunshine. On the 9th August the temperature in the shade was 100° Fahr. at Greenwich. The exposed portions of the south-east wall received the full effect of the sun's rays for many hours during the day, whereas the inner face was in contact with water pumped from the chalk some 200 ft. below ground level.

The difference between the temperatures of the two faces set up severe stress in the wall, causing unequal expansion therein.

Ejector Chamber in Concrete with Steel-plate Lining, Seaford.

(Fig. 36. Plan and Cross-sections.)

This chamber was constructed on low-lying land situated 200 yards from the sea. The subsoil was rock chalk, which as the excavation proceeded was found to be fissured, the subsoil water being in connection with the sea.

The original intention was that the chamber should be square on plan and of larger dimensions than the circular one actually constructed, hence the excavation was taken out rectangular, 19 ft. by 16 ft., the depth being 16 ft. below

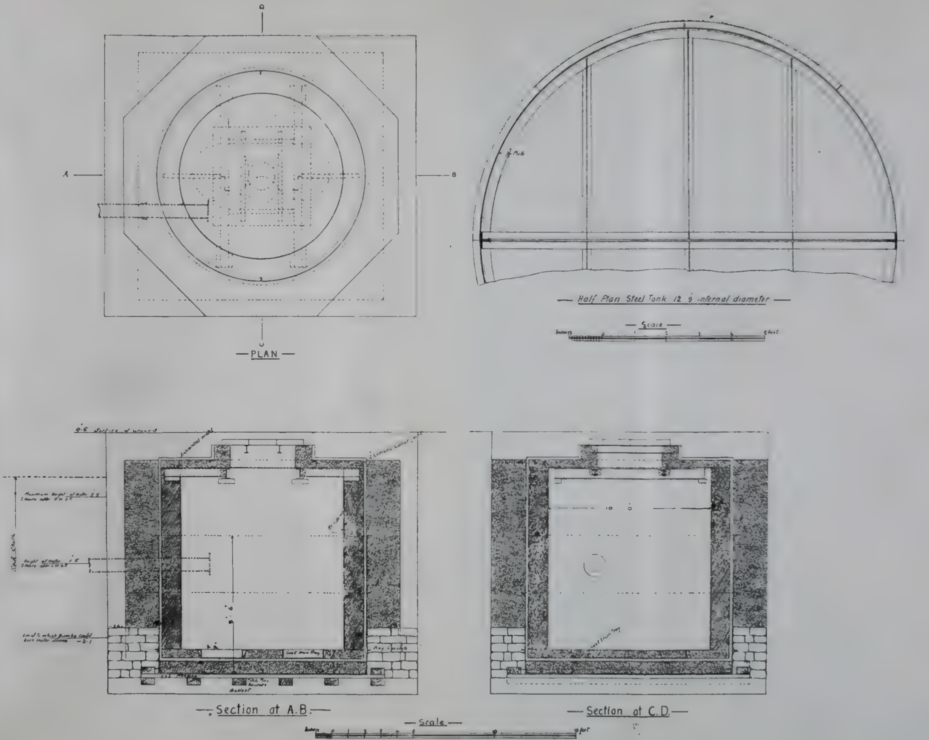


FIG. 36.—Chamber in Concrete with Steel-plate Lining, Seaford.

ground level. The levels to Ordnance Datum were as follows:—

Ground level ...	+ 9·5 ft.	and top of chamber.
Bottom of excavation	− 6·7	"
Floor of chamber ...	− 3·9	"
High water O.S.T. ...	+ 10·0	"
Low water O.S.T. ...	− 10·0	"

The excavation was carried down nearly 17 ft. below high-water level.

Two steam-driven 6-in. centrifugal pumps were installed to deal with the subsoil water, which was pumped up into the high-level sewers. The combined discharge of the pumps was 1,200 galls. per minute.

The principal trouble was from one large fissure, which, as the depth of the excavation increased, discharged a greater volume of water. This fissure could not be plugged, as the chalk at the sides would yield to the plug.

The subsoil water in the excavation acted in sympathy with the tide in the sea in the following manner. Under normal conditions of weather and sea the greatest height was attained about two hours after high water, and the lowest about two hours after low water. At spring tides the subsoil water reached a level of + 5·5 ft. above Ordnance Datum, and fell to a height of + 1·5 ft. During gales blowing inshore the water reached higher levels. Thus, on the 6th January, 1911, it attained a height of + 7·25, flowing over the walls then in course of construction.

The pumps could lower the water to a level − 3·10 and keep it at this level at high water O.S.T., and down to the level of the sump, − 7·5 ft., for a period of about two hours before and two hours after low water.

To have installed more powerful pumps would have been expensive, and in addition their effect on the fissure would probably have been to increase its yield in a greater ratio than that due to the increased head given by the further lowering of the water in the excavation.

It was therefore decided to construct the chamber by working during the hours the pumps could hold the water, and to shut out the same by placing a circular steel tank inside the concrete walls.

The construction of the chamber was carried out in the following manner:—The bottom of the excavation was

covered by ballast one foot in thickness to allow the water to reach the sump, which was sunk outside the excavation. Planks 10 in. by 5 in. were placed 2 ft. 9 in. centres in the ballast, so that their upper faces were level with the top of the ballast. On these was laid a close boarded platform 15 ft. by 15 ft., composed of 9 in. by 3 in. deals. This platform was strutted down to resist the water pressure underneath until sufficient concrete was laid to resist the same.

The circular tank rested on the platform, and was composed of riveted plates $\frac{3}{16}$ in. thick, stiffened by angle and T-irons; the internal diameter being 12 ft. 9 in. and the height 8 ft. 6 in. The tank arrived on the work in six segments, which were bolted up *in situ*.

The bottom halves to which the side walls for a height of 1 ft. 6 in. were riveted were placed in position and bolted up; then the remaining segments were bolted on, bag concrete being brought up round the outside to the level of $-2{\cdot}6$ ft., after which mass concrete was continued up to the top of the chamber, the minimum thickness of the surrounding concrete being 2 ft.

The bottom plate of the tank was covered by concrete 2ft. thick, in which two cast-iron trays were embedded, on which the ejectors were placed in order that their weight might be distributed. The interior of the tank was lined with 14 in. brickwork. A collar joint of 2 in. of cement was formed over the bottom and the sides, and the tank was covered with reinforced concrete, in which a manhole was provided for access to the chamber.

The concrete was composed of 200 lb. cement to $5\frac{1}{2}$ cub. ft. of sand and 11 cub. ft. of ballast, or $1 : 2\frac{1}{2} : 5$ mixture.

The work was satisfactorily completed in six weeks after the segments of the tank were delivered on the ground.

Concrete Mole forming Sewage Outfall into Sea, Seaford.

(Fig. 37, Longitudinal and Cross-sections. Figs. 38, 39, 40, Photographs.)

This outfall consisted of a concrete mole in which two lines of cast-iron pipes were embedded. It extended from the edge of the beach to below the level of low water O.S.T., the seaward end being 200 yards from the wall protecting the road from the sea. The site was an open bay in the

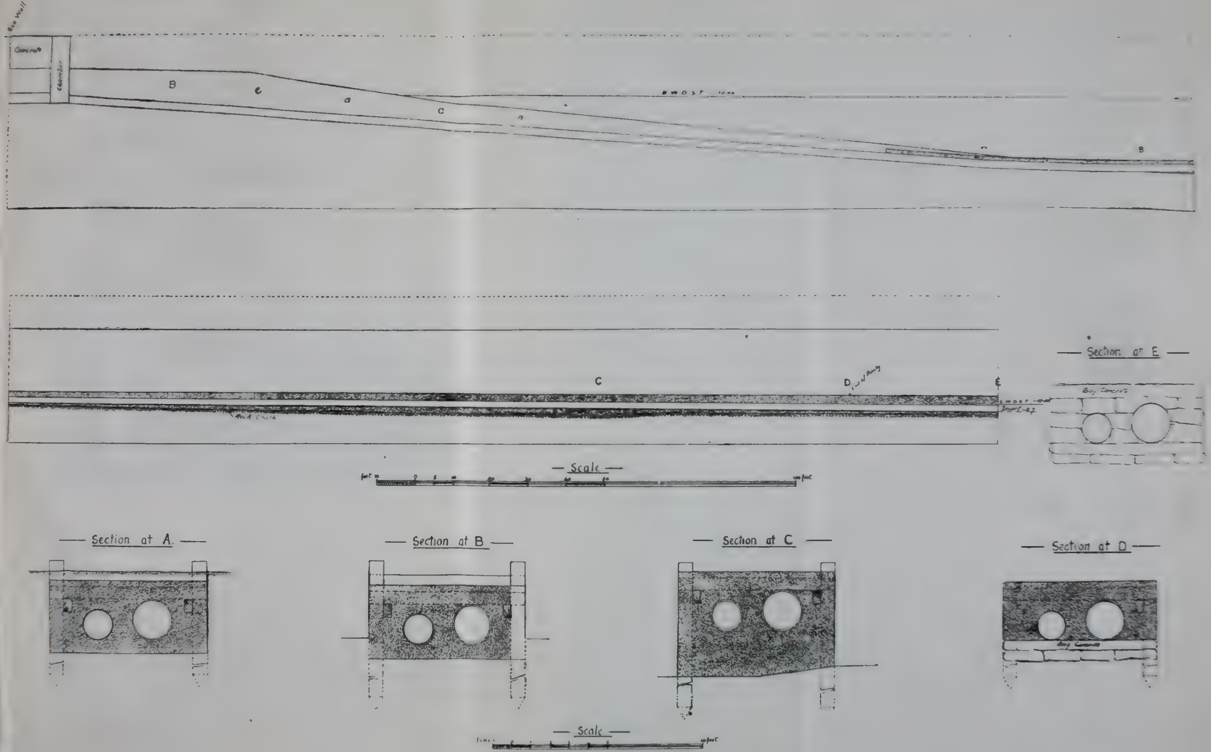


FIG. 37.—Concrete Mole Sewage Outfall into Sea, Seaford.

English Channel, exposed to the full force of the south-westerly and south-easterly gales.

Level of H.W.O.S.T.	... ...	+ 10 ft.
„ L.W.O.S.T.	... ...	− 10 „
„ Invert of pipes at Outfall		− 11·7 ft.
Length of Outfall from Sea Wall		187 yards.
„ Concreted portion	...	113 „
Width of „ „	...	8·5 ft.

For a distance of from 80 to 100 yards from the sea wall the foreshore is covered with beach, and beyond this distance

FIG. 38.—Seaford Outfall, November 1910.

with a layer of sand about 2 ft. in thickness, under which is rock chalk.

An outfall of 18-in. pipes had been in existence at this spot for many years, and extended originally for a length of 83 yards. In 1901 it was carried a further 67 yards by 24-in. cast-iron pipes. The pipes were held in position by oak piles driven into the solid chalk some 3 ft. deep, with oak cross-pieces to which the pipes were slung by iron straps.

In the spring of 1909 the travel of the beach over the top of the 18-in. pipe had worn a hole in it, through which beach gained access to the inside of the pipe, blocking it up.

The head of sewage thus engendered burst the weakened pipe.

This burst took place at 81 yards from the sea wall, where the beach ends and the sand is uncovered. Temporary repairs were effected and the pipes cleared.

Application had been made to the Local Government Board to re-sewer the town, and to extend and duplicate the outfall.

The contract for the work was commenced in the autumn of 1910, but for some time very little progress was made with the more difficult portion of the outfall.

FIG. 39.—Seaford Outfall, November 1912.

The work was done as tide work at spring tides, and even then it was not possible to carry it on unless the sea was calm; in addition considerable damage to plant and concrete was occasioned by gales. The outfall was completed in August 1913.

The work was carried out in the following manner: Rails were laid on the existing timber cross-pieces for the passage of trucks with materials. The sand was excavated down to the chalk, on which bagged concrete was brought up to the level of the undersides of the pipes, these being laid on the bag concrete. It was attempted to lay the rest of the concrete in this manner, but the attempt had to be given up, as it was

found the scour of the water along the unprotected ends of the bags at the sides and over the top of the outfall prevented the concrete from properly setting. Ultimately the sides above the bag concrete on which the pipes rested were close boarded, dry concrete being rammed in up to the level of the top of the cross-pieces and covered by close boarding nailed to the same. This plan answered admirably.

In part of the additional 100 ft. of length it was found impossible to drive the piles into the chalk; they were there-

FIG. 40.—Seaford Outfall, August 1913.

fore abandoned for the last 40 ft., in which length the concrete was brought up round and over the pipes in bags 8 ft. 6 in. long, filled *in situ* and well rammed. At the extreme end the concrete was filled into much larger bags in order to obtain weight to resist the wave action; these were laid in quiet seas.

The whole outfall has stood the winter gales well, the concrete being sound. In places several bags were damaged, but have been made good.

The concrete was 1 : 2 : 4 mixture, mixed dry by hand, placed in the work dry, and wetted by the percolation of the sea water. The cement was burnt in rotatory kilns,

ground all to pass 76 × 76 mesh, and residue on 180 × 180 not to exceed 10 per cent.; initial set not under 30 minutes, final set not under 5 hours nor over 10 hours.

Although not connected with concrete it may be of interest to mention that the 2-in. deals used in the timbering round the concrete were attacked by teredo worm, and rendered utterly useless in under 12 months' exposure. Specimens of the attacked timber are exhibited, two after 10 months' and one after 15 months' exposure.

Circular Hydrolytic Tanks in Reinforced Concrete.

(Fig. 41, Plan and Section.)

These tanks are designed and are about to be constructed for dealing with sewage on the Travis Hydrolytic Tank system.

The reinforcement is "Indented" bars, the stresses being limited to 16,000 lb. per sq. in. The concrete is 1:2:4 mixture. The shingle or broken stone is specified to pass through $\frac{3}{4}$ in. square mesh, and retained on $\frac{3}{16}$ in. The cement is to be burnt in rotatory kilns and to comply with all requirements of the latest British Standard Specification for slow-setting Portland cement, the quantity of cement in the mixture to be determined by weight. The stresses in the concrete are limited to 600 lb., and in direct compression to 500 lb. per sq. in.

The tank is divided into various compartments by concentric walls, and by radial division walls.

The central compartment is 12 ft. in diameter by 19 ft. 6 in. in depth. The sludge pipes from the other compartments discharge into the main sludge pipe through this compartment, which acts as a sludge well and valve chamber, and is always empty.

The next compartment, the hydrolyzing chamber, surrounds the valve chamber, and is 24 ft. in diameter, the bottom third being wedge-shaped, and the floor arranged in four ridges and four hollows, the sludge pipes being in the hollows.

The outer compartment is 60 ft. in diameter. The outer wall A B is vertical for a depth of 10 ft., and then inclined inwards B C to the bottom of the compartment, whence another wall D E inclined in the opposite direction extends to the wall K E of the hydrolyzing chamber, which it joins about one-third up from the bottom.

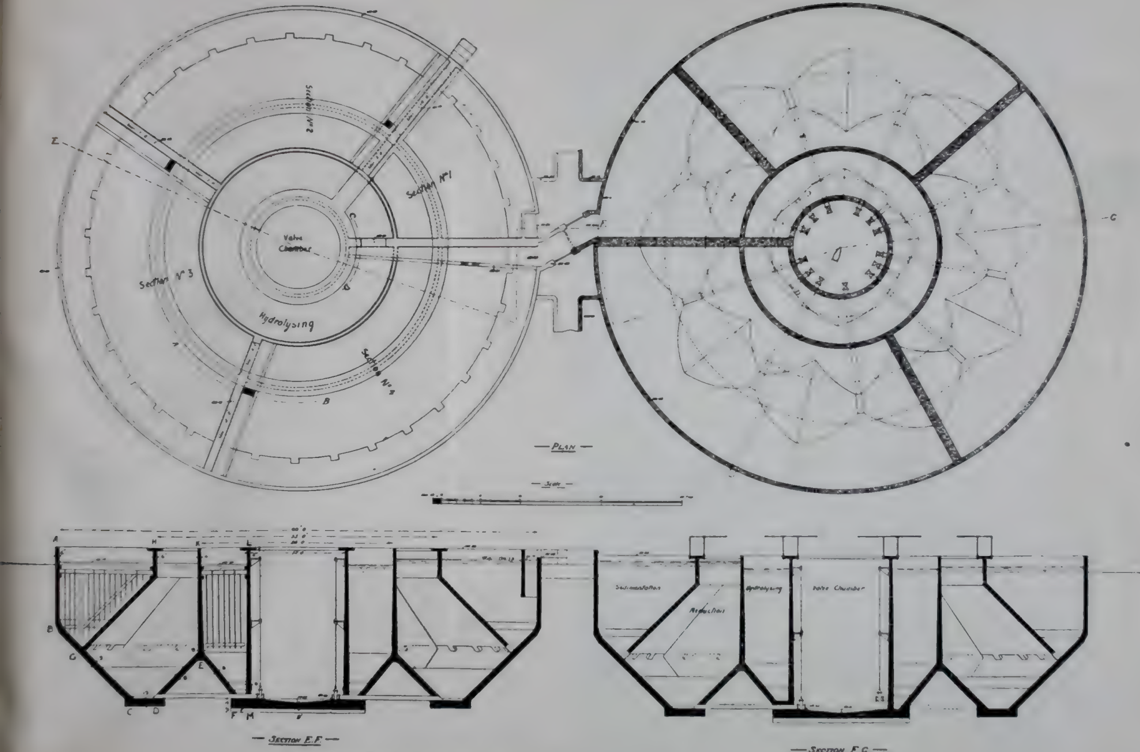


FIG. 41.—Reinforced Concrete Sewage Tanks, Dr. Travis' System: The Hydrolytic Tank Company.

From the outer inclined wall B C at about two-thirds of its height from the floor, is another concentric inclined wall G H —the angle between the two being 90 degrees—which divides the outer compartment into two parts, openings being left where the two walls meet. The outer part is the sedimentation chamber, the inner the reduction chamber. The openings in wall G H are 4 ft. long, and the length of the strips between the openings, which attach G H to B C, is 12 in.

The bottom third of the reduction chamber is triangular in cross-section, and the floor is arranged in ridges and valleys, with sludge pipes in the latter.

Radial walls divide the sedimentation and the reduction chambers each into four distinct sections.

The tank is sunk in the ground ; the levels being :—

Ground surface	130·0 ft. above O D		
Floor of tank...	112·5	,,	,,
Water level in tank according to section	130·7 to 130·2	,,	,,

When empty the outer walls A B, B C are stressed by the earth pressure, some of which may be transferred through the four radial walls to wall K E. In addition wall B C has to support the thrust of wall G H acting outwards. The annular foundation slab of concrete C D has to resist the thrust due to wall B C and wall D E, which latter partly supports wall K E. The circular foundation slab under the valve chamber has to resist the thrust from E F and weight of F L. Wall G H has to carry its own weight, and is, in fact, a truncated dome supported on a series of very short struts attached to B C.

When the tank is first brought into use, sewage enters the sedimentation chamber of the first section, and passes through the openings in wall G H into the reduction chamber. The sewage having filled the lower portion of the reduction chamber, rises equally in the two chambers until the level of the weirs is reached—on the radial wall dividing the first and second sections—the combined liquid passing by a channel into the second section. Thereafter the amount flowing through each of the chambers in the remaining sections is governed by the relative width of the controlling weirs.

From the time the first section is filled the radial division wall between this section and the second is under the pressure due to the head of liquid in the first section, until

the second section is filled, when the heads in each of the two sections being equal, the pressures on the radial wall neutralize one another.

The same effect is produced on the radial wall between the first and fourth sections, and remains in force until sections three and four are filled. The portions of the concentric wall K E, as the sections in the sedimentation and reduction chambers are filled in rotation, have to withstand the pressure of the water in those chambers. When the fourth section of these two chambers is filled, the sewage flowing over the weir from the sedimentation chamber passes out of the tank, whilst that from the reduction chamber passes by a submerged outlet near the top into the hydrolyzing chamber, which is filled and discharges over a small weir out of the tank. The liquid in this chamber puts the wall L F round the valve chamber into direct compression. The pressures on the walls A B and B C of the sewage in the sedimentation chamber, and of the earth outside, tend to neutralize each other.

Tanks of this description can be more economically constructed in reinforced concrete than in any other materials.

Similar tanks to the above are in use at Luton, the depth being 27 ft. and the reinforcement "expanded metal."

In conclusion the author wishes to express his thanks to the firms who have supplied information as to their manufactures, and to Mr. E. H. Moore, Assoc.M.Inst.C.E., for his assistance n the preparation of the paper.

APPENDIX.

DESCRIPTION OF MANUFACTURE OF CONCRETE TUBES FURNISHED RESPECTIVELY BY THE UNDERMENTIONED FIRMS.

MESSRS. JOHN ELLIS & SONS, LTD., Barrow-on-Soar, Leicestershire.

(Figs. 1, 2, 3, 4, 5, and 9.)

The cement used in the manufacture is made by the firm from the lower beds of the Blue Lias limestone. The raw stone is brought from the quarries to the cement works and deposited on separate floors according to the bed it was taken from. Stones from the various floors are mixed in such proportions that the aggregate formed by them contains 76 per cent. of carbonate of lime. The assorted stones are then ground to pass a sieve of 20 by 20, moistened and pressed into bricks, which, after drying, are burnt, with coke, to a clinker in a "bottle" kiln, the temperature being 1,600° C. The clinker after withdrawal from the kiln is picked over by hand, and any underburnt is discarded. The thoroughly calcined clinker is passed through a crusher, then through stones, and finally through a tube mill until the cement passes 180 by 180 sieve with 15 per cent. residue. After aeration the cement is ready for use and is guaranteed to give a tensile breaking strain of from 600 to 700 lb. in seven days, with 10 per cent. increase in twenty-eight days. When gauged 3 to 1 with sand (granite cracker dust) 300 lb. in seven days, and 350 lb. in twenty-eight days.

For making tubes, three parts of Leicestershire granite and sand, which has all been washed and passed through a ¼-in. sieve, are, with one part of cement, mixed dry in a mixing machine, after which sufficient water is added to the mixture in the pan, and the mixing process continued until the whole is thoroughly moistened. The damp mixture is then placed in vertical steel moulds and rammed by a mechanical rammer until the whole tube is consolidated. If reinforced, three rings of steel ¼ in. in diameter are placed in each tube. The tubes remain in the moulds about two or three days until they are fit to handle, and are sent out of the works after six months in store. If required the tubes are placed in a bath of 2 per cent. of silica.

When bedded in concrete up to the springing, the reinforced tubes are guaranteed to carry a load of 8 tons per 3 ft. length, and the ordinary tube 6 tons, load to be applied to a 6-in. beam extending the length of the crown of the tube.

Sizes manufactured.

Circular	12 in. to 60 in.	Thickness	... 1¼ in. to 5 in.
Elliptical	18 in. by 24 in. to 60 in. by 40 in.	"	... 1¾ in. to 4 in.

In May 1914 six tubes were tested for resistance to thrusting stress by Messrs. Kirkaldy & Sons, with the following results :—

Tubes 30 *in. inside diameter by* 3 *ft. long, thickness from* 2·43 *in. to* 2·63 *in.*

	Stress per foot run.	
Three tubes tested plain concrete	1,873 lb. average	Test applied by 2 in. steel-bar on rubber bearings along crown of tube. No lateral support. Tube resting on 4 in. rubber.
Three tubes tested with three steel ¼ in. hoops	2,348 lb. average	

The tubes were made of the ordinary mixture, and were tested when six months old.

Samples of concrete are exhibited, together with samples of materials used in its manufacture, as follows :—

1. Limestone.
2. Shale.
3. Shale and limestone in correct proportions.
4. Clinker.
5. Cement.
6. ¼ in. washed granite and sand as used.
7. Small pieces of granite concrete.
8. Cube of granite concrete cut from a 24-in. steel-ribbed tube.
9. Cube of granite concrete cut from a 36-in. non-ribbed tube.
10. Cube of granite concrete cut from a 48-in. non-ribbed tube.

Messrs. Sharp, Jones & Co., Parkstone, Dorset.

(Figs. 6, 22, and 23.)

The cement used in the manufacture is purchased. At the present time it is made from chalk and Medway clay, calcined in rotatory kilns, and ground to leave only 2½ per cent. residue on 180 sieve. The Le Chatelier test, 24 hours aeration, gives 1 millimetre expansion.

The tubes are made from a mixture of 1 part of cement to 3 parts of crushed Guernsey granite with sufficient clean, sharp white sand. The granite is found in solid masses of great thickness, being free from admixture of clay, etc.

As regards the strength of rock concrete tubes, the following is the result of a test recently made by Messrs. D. Kirkaldy & Son, to ascertain the resistance to thrusting stress of four tubes, two of which contained in each foot of length a metal hoop ¼ in. diameter, the other two being plain. The tubes were 18 in. diameter, 2 ft. long, 1½ in. thick, and several years old.

Stress per foot-run in lb.

With hoops.	Without hoops.
2,175 average.	2,232 average.

These pipes were in every respect as far as possible identical, having been made consecutively by the same men, on the same day, with the same machine, and the same materials. The test was what is known as the knife-edge test, the pipes being compressed between two parallel plane surfaces with neither cradle nor saddle, nor any lateral support.

Samples of concrete are exhibited, together with samples of materials used in its manufacture, as follows :—

1. Portland cement.
2. Sand.
3. Crushed Guernsey granite.
4. Cube cut from granite concrete cube.
5. Specimen of vitreous stoneware and Portland cement concrete, age 35 to 40 years.

Messrs. Hughes and Lancaster, Ltd. (*Licensees for the "Bonna" Pipe*),

16 Victoria Street, London, S.W.

(Figs. 10, 11, 12, 13, 14, 15, 16, and 17.)

The steel tube is usually made up from three sheets of steel, the joints being welded by the oxy-acetylene blowpipe. The length is usually 10 ft. The external reinforcement is composed of cruciform helicals held together by longitudinal round bars kept in position by steel wire. The internal reinforcement is made in the same way, but with helicals and longitudinals of smaller section. The two sets of reinforcement are slipped respectively over and into the steel tube, and the steel part of the pipe is then ready for the concrete.

The assemblage of tube and cages is erected on a table, and a collapsible steel core is placed in position inside, and a steel mould in three stages is placed outside the assemblage, leaving the requisite annular spaces to be occupied by the concrete.

The concrete is composed of two parts of sand and one of cement, which are hand-mixed—by the aid of special paddles—in a tilting trough carried on an elevated travelling gantry. When the mixing is completed the concrete is run into the spaces between the core and the moulds. During this process the moulds are sharply struck with wooden mallets to drive out the air from and to consolidate the concrete. The pipes are kept in the moulds until the concrete is set sufficiently to allow of their removal, after which the pipes are left standing on end for about a month to harden and then removed by a crane and laid in store.

The collars placed round the joints are made in a very similar manner to the pipe itself, being proportioned to the working pressure in the main. In cases of low pressure, a preparation of bitumen is first run into the space provided at the end of each pipe and secured until sufficiently hard by a temporary steel collar. The collar is removed and the skeleton of the joint, made of the same material as the external reinforcement of the pipe, is slipped over the bitumen joint and enclosed in a mould into which the concrete is run. For high pressure the bitumen is replaced by lead and tarred gasket, caulked underneath a permanent corrugated steel collar and between it and the steel tube of the pipe ; when this is done, the reinforcement and moulding of the joint is completed as before.

Specimens of a small tube and collar are exhibited which show the reinforcements. Portions of a Bonna pipe 21 years old, which was removed after transmitting Paris sewage for 13 years, are also exhibited and further referred to in the paper above.

MESSRS. SIEGWART, LTD.,
Gwydir Chambers, 104 High Holborn, London, E.C.

(Figs. 18, 19, 20, 21, 42 to 49.)

Two methods of manufacture of Siegwart reinforced pipes are in use at the Company's Works at Grays.

In the first method the process may be divided into three operations: (1) making the concrete laths, (2) making the pipe, (3) asphalting.

(1) *Making the Concrete Laths* (Fig. 42).

A wood frame is made of the required length and width, equal to the circumference of the pipe about to be made. This area is divided

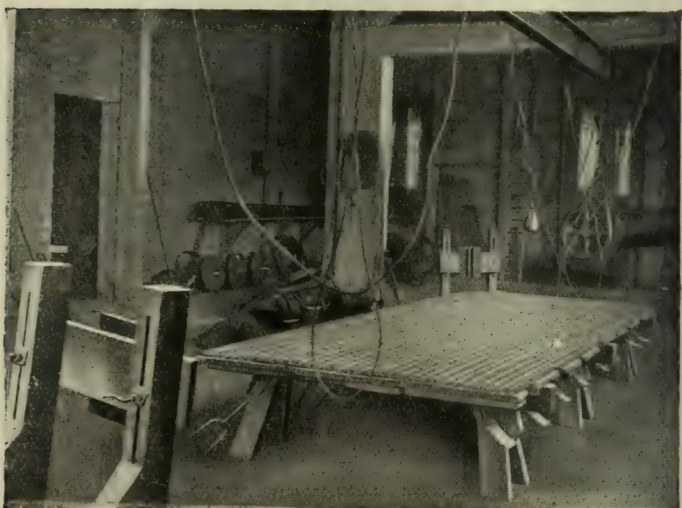


FIG. 42.—"Siegwart" Concrete Laths on Jute Foundation.

by wood or iron strips of the thickness of the required laths and about ⅜ inch wide. This frame is placed on a sheet of loosely woven jute on a prepared flat floor, and fine concrete, 1 to 3, is spread over the entire surface. The frame, with the wood or iron strips, is then removed, leaving the concrete laths adhering to the loose jute foundation. The laths are about 1½ by ⅝ in. and of the required length for the pipe, with spaces about ⅜ in. wide between them.

When the concrete has set the set of laths are rolled into a bundle and removed to the pipe-making shops, where they are spread out on a table. The longitudinal reinforcing rods are then placed in the spaces between the laths, and a layer of concrete is spread over the

FIG. 43.—"Siegwart" Concrete Laths being rolled round Steel Core.

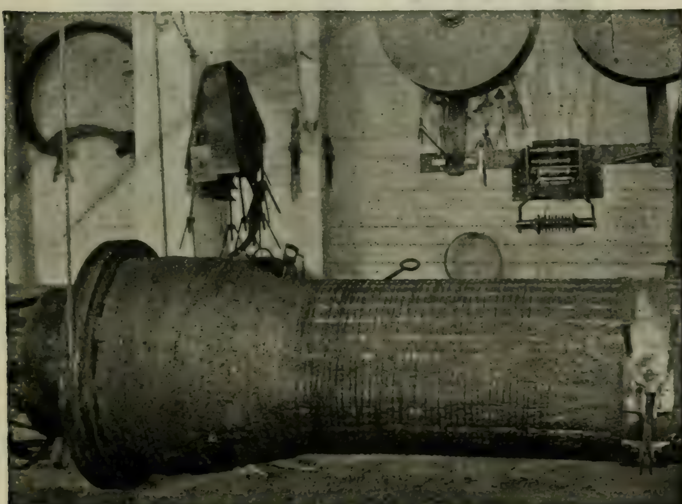


FIG. 44.—"Siegwart" Helical Reinforcement being wound on Concrete, followed by Concrete Covering.

entire surface, filling the spaces between the laths, and thus surrounding the rods.

(2) *Making the Pipe* (Fig. 43).

The collapsible core is then brought up and the sheet of prepared laths is rolled round it and secured by straps, which are subsequently removed.

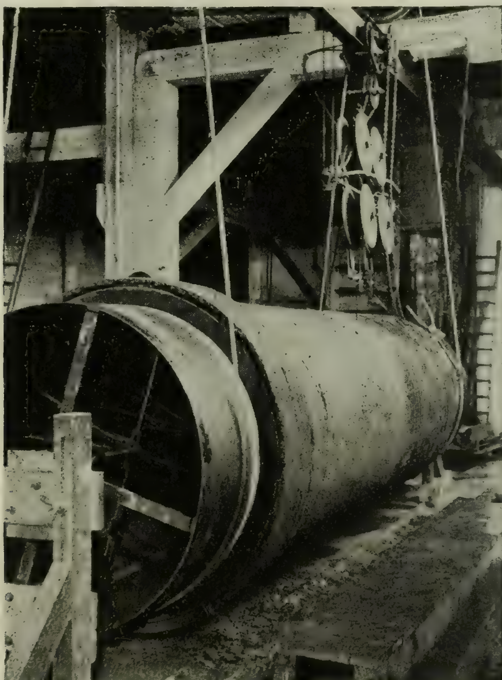


FIG. 45.—"Siegwart" Helical Reinforcement being wound on Concrete, followed by Concrete Covering.

The core with the surrounding laths is then suspended in slings, wires for the spiral reinforcement are attached to one end of the laths and the whole made to revolve at a convenient speed. Concrete is spread on in a fairly dry state during the revolutions, and at the same time the wires are bound round and guided so as to have a uniform pitch to the opposite end of the pipe ; a second, third, or even fourth layer of wires can be put on by simply reversing the direction of the wires without stopping the machine, concrete being spread on all the time (Figs. 44 and 45).

FIG. 46.—"Siegwart" Concrete covering Reinforcement, with Canvas Webbing being wound over Concrete.

FIG. 47.—Back of "Siegwart" Long Pole and Pipe Machine.

When the requisite thickness of concrete and layers of wire are obtained the machine is stopped, the ends of the wires are finished off, wrought iron rings are fixed and the concrete made good at both ends.

Canvas webbing is wrapped round the pipe, Fig. 46, which can then be removed to the yard. On the following day the core is removed and the pipe is left to harden.

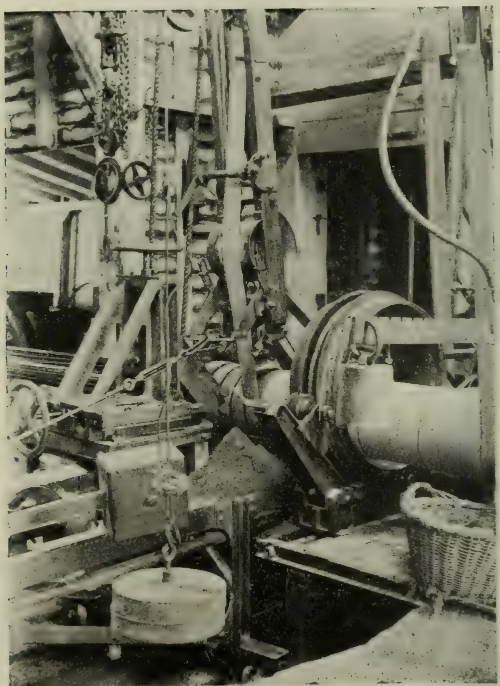


FIG. 48.—Detail of "Siegwart" Pole and Pipe Machine, showing wire-woven band applying concrete and canvas band. This canvas band, after compression of concrete by rollers, is unwound to allow of helical reinforcement being bound on; the canvas band is simultaneously re-wound to hold concrete in place.

(3) *Asphalting.*

When sufficiently matured, the pipe is placed in an oven and thoroughly dried. Prepared end fittings are then attached and a sufficient quantity of a mixture of asphalt and bitumen poured into the pipe, which is revolved at a high speed, the result being that the mixture is forced, by centrifugal action, to adhere to the concrete, and to spread over it in an even, smooth layer.

In the second method of manufacture (Figs. 47, 48, and 49) the steel core is fixed so that it can be revolved between two travelling carriages and be moved horizontally over the special machinery, consisting of the trough for the concrete, out of which the necessary quantity of concrete is conveyed on to an endless chain in the form of a channel in which a canvas band lies. The concrete is delivered on to the band, there being an attachment which regulates its thickness. The band with the concrete on it is helically wound round the core and is then compressed between two rollers. By this process the concrete is solidified. The steel wire reinforcement is then wound on helically, the canvas being simultaneously unwound. During the winding on of the wires they are continuously

FIG. 49.—"Siegwart" Long Pipe being removed from Machine.

painted with cement grout. The longitudinal reinforcement, which is assembled together beforehand, is then placed over the helical wires, and the whole is covered by a thin sheet of concrete, laid on by the band as in the first instance. The interior of the pipe is coated with asphalt and bitumen.

The concrete is composed of 1 part of Portland cement to 3 parts of aggregate—flint grit and washed sand—screened through ¼ in. mesh. The concrete is machine-mixed.

Samples are exhibited, together with samples of materials used in their manufacture, as follows :—

1. Washed sand.
2. Flint grit.
3. Specimen of reinforced concrete tube 12 in. diameter.
4. Broken specimen of reinforced concrete tube showing construction.

DISCUSSION.

THE PRESIDENT (PROFESSOR HENRY ADAMS): —This paper is one that is likely to be extremely useful to any of our members engaged in municipal engineering. The author tells us that the concrete should be machine-mixed, and I should like to know whether he is not in favour of batch-mixing for the concrete, as distinguished from continuous mixing. With regard to the tests on the concrete tubes on page 187, most municipal engineers have been on the watch for many years for experiments on the strength of pipes or sewers under load from above, and this refers to an experiment of the kind. It appears from this that about 200 lbs. per inch run was sufficient to fracture the tubes. That was direct upon the upper portion of the tubes, but it must be remembered that in practice the tubes would not be short, unsupported lengths, but would be continuous, so that however far the load may extend and affect the pipe, the pipe itself is supported by a portion beyond, so that the strength in practice would be greater than the strength under test. Then, again, the load always comes upon the surface of the ground some distance above the top of the sewer, and in passing through the ground the pressure is spread outwards at an angle varying, according to the different authorities, from 30 to 45 degrees from the vertical at the point of application. That, again, reduces the intensity of the pressure by the time it reaches the pipe. But all experiments relating to this matter are of value and subject to further investigation. On page 195 the vibratory movement is referred to as being particularly useful in forming the Jagger concrete tubes. I have had experience of the vibrating apparatus in connection with artificial paving, and there it was found extremely useful in consolidating the material and caused it to withstand wear and tear for a longer period. On page 198 I see that the Bonna pipe is exposed to currents of hot air, but I think the experience of most engineers and architects with reinforced concrete is that it requires some protection from hot air—moist cloths over your test briquettes and matters

of that kind—so that one would think they are rather sacrificing the strength of the pipes to shortening the time for their manufacture. In the Siegwart pipes I notice there is a cruciform steel bar for forming the helicals. I should like to know if the author can say why that should be a cruciform section. I prefer the simplest section that can be obtained, plain round or square; but there may be some reason in this case for adopting the cruciform section. With regard to the failure of the reservoir on page 213, we are not told where it is or who was the engineer; but any one in the habit of designing arches and abutment walls, looking at that diagram, would see instantly that it ought to fail; it was bound to fail; there is not sufficient resistance in the abutting walls. Another somewhat similar failure came to my notice where the arches spread and gave way, but that was due to the earth filling on top being carried over some of the arches complete and leaving others without any load at all, so that naturally they yielded. It seems to me that in filling the earth covering over reservoir arches it is desirable as far as possible to begin at the springing of the arches and carry it gradually along, so that the thrust is balanced all the way through. On page 218 the concrete in the floor was composed of cement and fine furnace slag or some kindred material. No doubt it was furnace slag if it gave off a strong smell of sulphuretted hydrogen. A case came to my notice recently where a strong smell was given off from the concrete at certain times, and it was found that coke breeze and furnace slag and material of that kind had been used. The bad material may have been through only a portion of the work. On page 219 there is another unfortunate reservoir. In that case if you look at the section you will observe that the whole of the batter of the external walls is on the inside. That means that the centre of gravity of the wall is thrown towards the outside of the wall, and the wall offers very much less resistance to the thrust of the arches than if it had been turned the other way; and I cannot see any reason why it should not have been turned the other way, but that is a point that seems often to be overlooked.

Mr. T. J. MOSS-FLOWER, Assoc.M.Inst.C.E., F.R.San.Inst.:—With regard to the ejector chamber that was put down on the seashore, it struck me as being rather an expensive method of construction, and one would think it would have been better to put in some cast-iron tubing, which could have been done much more expeditiously. With regard to the sea outfall that Mr. Tingle described, I would like to ask him how long an interval he allowed to elapse between putting in the concrete and the return of the tide. I was interested to hear that by close boarding the sides of the trenches and filling with concrete and then covering the boards over success was obtained, because in a similar experience one found that it was not very successful: the tide seemed to get in and destroy the concrete. I agree entirely with the President that the design of the first reservoir shown upon the screen is such that it could not do otherwise than fail. With regard to the second reservoir shown upon the screen, the President referred to the batter of the walls being formed on the inside; but one has seen recently a case where the batter was thus arranged because it was assumed that there would be a greater pressure from the outside than could be thrown on the walls by the pressure of the water on the inside; but in that case the precaution was taken to see that the steel reinforcement at the top of the side walls was tied to the reinforcement in the arches covering the reservoir, and there could be no possibility of the water forcing the walls of the reservoir out. One can see by proper engineering and taking care in the design of one's reservoir if there is going to be a greater thrust on the outside, and if so it might be an advantage to have the batter on the inside. In normal conditions one would expect to see the batter on the outside, but it is just a question as to whether you want the greater strength on the outside to prevent the walls being turned out, or vice versa. One would think in the case that has been dealt with that it would have been better to have the batter the other way. As regards the first reservoir mentioned and which failed, the arches were altogether too flat. I think, perhaps, sometimes in forming the roofs of our reservoirs we

forget that the covering on the top is liable to become saturated with water, and oftentimes there has not been any provision made to get the water away from the spandrils, and in consequence considerable extra weight is thrown upon the roof of the reservoir. I am at present engaged in using a good many concrete tubes described by the author, and one wonders really how it is that that type of pipe is not brought into greater use. There is one interesting point brought out in the paper, and that is in regard to how much stronger the pipes made with an aggregate of ground-up stoneware are than those made with an aggregate of gravel or other material more generally used. The author also brought out a very interesting point in connection with the small term of years for which a loan could be obtained for reinforced concrete. In consequence of this short period one oftentimes finds it is infinitely better to use mass concrete in the place of reinforced concrete. I will take a case which I am dealing with now. I intended to use reinforced concrete, but we found the stuff for our mass concrete, our sand and gravel, and so on, right on the spot, and having regard to that and the fact that our loan could only be spread over a period of fifteen years, I found that it was in the interest of my clients to use mass concrete instead of reinforced concrete. We all know there is a great deal of grumbling with the Local Government Board on the point that they will not allow loans for this class of work to be extended over the same period as is allowed in the case of ordinary concrete; but, of course, the Local Government Board have to deal with public money, and they must allow a large factor of safety, and I think myself that that public body has been quite right up to the present time. I do not mean to say that in the near future they will not be justified in altering their views; but there is safety in going slow, and I think they have been quite justified in the stand which they have taken up. In reinforced concrete one has to depend upon the human element so much, and unless it is very carefully looked after, or if you go cheeseparing and make your concrete too thin, you incur grave risks. I hope you will pardon me for quoting a case of my own again,

but recently I had an elevated reinforced concrete reservoir put up in a very exposed position, and when the wind blew and the rain came down at night and I woke up and thought of it I could not help thinking there was a possibility that we had been steering too near the wind and that it might come over. The work was well supervised. The clerk of works was provided with a tent and was on the site day and night during the whole time. It was put up eighteen months ago and has stood some very rough winds, and I am sure it is all right; but if we had not taken very great care there was a danger of it not being so.

PROFESSOR STEPHEN M. DIXON, M.Sc., M.A., M.Inst.C.E., M.C.I.:—There is only one point that I can think of for the moment upon which I should like some further information. That is the question with regard to the pipes which are under the high pressures which are given in the paper at Swansea and Norwich. How were the joints of those pipes made? People sometimes have a good deal of trouble with percolation of water through concrete. When the concrete is made with very great care that can be avoided; but I think it must be very difficult to make a joint which will stand pressure such as that mentioned in the paper. Does percolation take place in those cases?

MR. T. C. DAWSON, M.C.I.:—With reference to the point about the internal batter of that reservoir, as the pressure is normal to the surface the line of pressure would not be horizontal, but it would be inclining downwards, which would assist the stability of the wall so far as the water pressure is concerned.

MR. D. B. BUTLER, Assoc.M.Inst.C.E.:—Referring to the Local Government Board conditions, and their only allowing ten years for reinforced concrete and twenty years for ordinary concrete, where in contact with water or sewage, Mr. Moss-Flower says they are right in going slow. Well, perhaps they are, but if reinforced concrete is properly made water cannot detrimentally attack it. Turning next to the tests given on page 187, it is rather a pity that Mr.

Tingle has brought in tests and results which are more than thirty years old. Cement nowadays has so vastly improved that they are quite misleading. Moreover, the tests given are, I understand, made with broken stoneware, a kind of waste product from pipe manufacture, and therefore scarcely available on a commercial scale. Of course, broken stoneware might be expected to give a much better strength than ordinary gravel, because the former would have a sharp, angular, jagged section, and would give a better tooth, as it were, for the cement to adhere to than the more or less smooth and rounded gravel.

On page 189 I see Mr. Tingle quotes a Mr. Butler. May I, in the first place, be allowed to say that this Mr. Butler is in no way related to me. Mr. Butler's statements are, however, again thirty years old, and in that particular paragraph are a libel on the present British manufacturers.

THE PRESIDENT :—It is thirty-five years ago, you must remember.

MR. BUTLER :—Exactly. Therefore I think it is rather a pity that it should be brought in now. He suggests there that the English cement is not so good as that made at Stettin and Boulogne owing to inefficient control. I have visited nearly all the English cement works, and also the Belgian and Boulogne cement works, and I have also had cement through my hands for testing from all the chief factories in Belgium and Germany, as well as other continental countries, and I say, without fear of contradiction, that the English cement of the present day is, if anything, superior to the German and foreign cements. When the paper in question was written, thirty years ago, perhaps I could not have said that. If I were asked then which was the best cement, German or English, I should have been compelled to admit that the Germans beat us ; but now our methods have improved so much that, on the average, English Portland cement is far superior to any foreign brand. I say that, having had many thousands of samples of foreign cements through my hands in the last few years. With regard to the reservoir which failed, I

could not quite gather from the paper how much of that floor was covered with the slag and why it was so covered. Perhaps Mr. Tingle will kindly tell us. Probably the reason why the cement round about was disintegrated was owing to some chemical reaction between the sulphur compounds in the slag and the lime in the cement. I have come across several similar instances. I recently had to investigate and report on a sewer into which was percolating the waste from an alkali works, which contained sulphuretted hydrogen in solution, and the result was that the mortar became thoroughly rotten and the sewer had to be relaid altogether.

With regard to the Seaford matter, the only specification is as to fineness and setting time. Surely there must have been some stipulation as to strength, and more especially as to the all-important matter of soundness. In testing cement the soundness is the first thing, because naturally if the cement is unsound and subsequently expands it is worse than useless as a constructive material.

MR. CHARLES SHARP (of Messrs. Sharp, Jones & Co.) :—I have taken the opportunity of referring to the paper that was read before the Municipal Engineers in 1879 by Mr. Fletcher Woods, and that shows that concrete applied to sanitation is not a new matter by any means, but it is one which it has been a laborious matter to get engineers to believe in, because in old days concrete was regarded as a more or less coarse material for use in bulk. It was used for paving, it is true, forty years ago and more, but it was not used for such delicate things as drain pipes. Mr. Butler has just been making some observations on those old tests of concrete pipes made with broken stoneware. I should like to confirm what he says as to the quantity of broken stoneware available for such a purpose being in these days practically none at all, because stoneware in the strict signification of the term means, as most members here know, ware made from the clays of the Tertiary formation, and not ware made from clays found in the coal measures. The latter are burnt at a very much lower temperature and cannot stand anything

like the high temperature of the clays from the Tertiary formation, commonly known as the Poole clays and the Newton Abbot clays, and therefore it would not be a safe thing to use as a raw material for concrete tubes broken ware of pipes made from the clays of the coal measures, as they certainly would not have anything like the density, strength, and non-porosity of tubes made from broken stoneware. Broken stoneware is practically in these days not obtainable. It is all ground up again and mixed with the fresh clay in order to improve the quality of the pipes.

On reference to Mr. Fletcher Woods' paper, I find that there are some other tables of tests given, and one in October of 1878. I was present at that myself, and well recall it. Five concrete tubes, 2 ft. long and $1\frac{5}{8}$ in. thick, were tested in a testing-machine between two surfaces without any support, and they showed an average of over 3,000 lb. breaking strain on a 2 ft. length, which for a pipe 2 ft. diameter and $1\frac{5}{8}$ in. thick is really a result that compares most favourably with tubes made at the present day, of which you will find examples in the Appendices to Mr. Tingle's paper. There is also a case of a 24-in. tube, 2 in. thick, that broke under 4,700 lb. That was 2 ft. long, and it gives 2,350 lb. breaking strain per foot run, and that also is fully equal to what is obtained at present. The age of the tube is not stated, but I know that 24-in. tubes were not made 2 in. thick until quite a short time before. It seems to me that that shows two things. One is that the broken stoneware pottery is a most excellent aggregate if it is obtainable, and the other is that where, even in those far-off days, one took the trouble to obtain good cement, good cement was obtainable ; but perhaps there was not in general the demand for it. I take it the people were not so strict in their specifications, but where it was demanded it was possible to obtain a good cement. As regards the observations about the Local Government Board's regulations, I cannot help sharing Mr. Moss-Flower's view, if I may say so, for this reason—although, not being an engineer, I rather hesitate to express an opinion—but it strikes me that, although people say the War Office and the Board of Works and the

Admiralty are all using reinforced concrete and why should not the Local Government Board allow it also, there is this to be remembered, that the Local Government Board has to sanction loans for money to be expended most frequently under the supervision of the local authorities, and the officials of those local authorities are not all of them of necessity men of such wide experience as those whose services the War Office and the Admiralty and the Board of Works can command. They are not as a rule Westminster engineers, for instance. It is clearly the case that concrete does want the most close supervision, and it is a thing one knows very well when one is making it day after day and week after week, as my firm have been doing now for forty years. We know that the men who are at the work must be constantly looked after, that cement is a chemical article, and that unless the greatest supervision is observed at every single stage of the manufacture you cannot obtain a good result. I take it that what one finds is the case in the manufacture of concrete tubes is, in a measure, the case in using concrete in any form at all, and unless a local authority, having received sanction from the Local Government Board to use reinforced concrete, is able to command the most accurate and careful supervision over the use of the concrete, it seems to me that the concrete industry is likely to receive a severe setback through serious accidents occurring in the future, so that I am quite with Mr. Moss-Flower in the observations he has made on that point. I think that the reference on page 187 to those old experiments was intended—so I took it—to show that the application of concrete to sanitation really dates back a large number of years. It is forty years exactly this year since we introduced the manufacture of concrete tubes, and therefore I think that that was to explain how old the use of concrete for sanitation is. As regards the Stettin cement, I quite agree with Mr. Butler that even in those days good English cement was obtainable. I was looking up some old records the other day and I found that we made a trial of some cement from Stettin, but I observed that it was not kept on with, so that that would rather tend to show that even

so many years ago—thirty-five at the least—English cement, where you asked for a good article and insisted on getting it, was to be preferred. In these days I am quite sure that it is impossible to beat English cement. There is a point that I think the Concrete Institute, if I may respectfully say so, should endeavour to bring its views to bear upon, and that is—I speak subject to correction—as to the great importance of fine grinding. It seems to me that although the standard specification has specified 18 per cent. on the 180-mesh sieve, that is a specification which requires tightening up considerably. I am certain, so far as my practical experience goes, that it is economical to pay a little extra and get a more finely ground cement. For instance, when you are buying coal you know from your domestic experience that it does not pay to buy coal on which perhaps one-half of the value represents railway carriage, if a large part is slack or refuse, and, in a less degree perhaps, it seems to me that it does not pay to buy a cement in which a large proportion is coarse particles, which, if they have cementitious value, do not have that cementitious value until a very long period afterwards ; and therefore I think it is better to pay a little extra money and get a larger proportion of perfectly useful material.

MR. S. BYLANDER, M.C.I. :—Regarding the manufacture of pipes, I should be glad to know what provision is made at the joints to take up the axial force produced by the pressure of water.

MR. W. A. GREEN, M.A., B.Sc.Eng., Assoc.M. Inst.C.E., M.C.I., also expressed his thanks to the author.

MR. W. CAMPBELL SHARMAN (of Messrs. John Ellis & Sons, Ltd.) :—There is only one thing that struck me in the discussion with regard to concrete, and that is that so little has been said about the aggregate. We have been talking about the cement, but there has been very little said about the aggregate with which the cement is mixed, and I think the graduating of the aggregate is quite as important as

anything else with regard to concrete. I have the greatest belief in a perfectly graduated aggregate, and that is a thing that one is very seldom able to get. I have heard nothing mentioned with regard to a granite aggregate, and I should like to have had some expression from the meeting as to the difference between the aggregate that has been suggested (gravel and broken earthenware) and granite aggregate. I believe most of the cement that is used is tested with Leighton Buzzard sand. I had, a little while ago, an opportunity of making some experiments, and I reduced a granite sand to the same specification as the Leighton Buzzard sand. I will read the result. The same sample of cement was taken, and it was mixed up 3 and 1, 3 parts of sand and 1 of cement. The 7 days' test with the Leighton Buzzard sand showed an average of 300 lb. per square inch; the 28 days' test showed 434 lb. per square inch. We then took some granite sand and reduced it so that it all passed through a sieve 20×20, and it was all retained on a sieve 30×30; it was mixed 3 parts of granite sand to 1 part of cement. The result was that in 7 days we got 486 lb., 3 and 1, and in 28 days 654 lb. on the square-inch section. I think that proves very clearly that quite as important as the cement is the question of the aggregate, and I suggest that more attention should be paid to the aggregate in the future than in the past has been the case.

MR. ALLAN GRAHAM, A.R.I.B.A., M.C.I.:—The President, referring to the cruciform reinforcement in the Bonna pipe, asked the reason for this arrangement. Every one knows the difficulty in placing the circular reinforcements round a sewer in the proper position. To obviate this the cruciform type was adopted, which, while giving the proper area of metal at the exact required spot, acts at the same time as a shear member and distance piece. This can be verified by an examination of Fig. 10, which shows the arm of the cross where the tensional metal is provided as far away as possible from the longitudinal reinforcement. While considering the reservoir that failed, attention was drawn to the fact that the batter was on the inside. Probably this was done

to take advantage of the fact that water being a fluid always presses normally against the surfaces opposed to it, although the batter slightly increases the head of water to be calculated for ; but, on the other hand, the line of pressure inclined downwards reduces the length of the arm, and consequently equally reduces the overturning moment. Of course, in the case of retaining walls for earth it would certainly be better to place the batter on the outside.

In carrying out reservoirs the necessity has been shown to provide plenty of distribution rods to take care of the temperature stresses, as witness the behaviour of Hadham Reservoir under the action of extremes of temperature.

With regard to the failure of the large reservoir, the spans were evidently too long for the thickness of the wall provided. The buttresses provided in the reconstruction demonstrate that ; but there is little doubt that the upward pressure of the water being 1 ft. 8 in. higher than was intended acted on the roof arch and thrust out the wall head ; the failure was thus not altogether due to the wall, but to the pressure of the water pushing against the domed top.

MR. ROBERT N. SINCLAIR, M.C.I. :—I must say that I endorse the remarks of Professor Adams. I am disappointed at not hearing details as to the composition of the concrete. It would be very useful if Mr. Tingle could have given us the grading of the ballast and sand from which this concrete was made. Nowadays the porosity of concrete, with those having ferro-concrete work in hand, is a very important point, and it would be very interesting if Mr. Tingle could give us some experiments as to this.

MR. TINGLE :—Which concrete do you mean—any special one?

MR. SINCLAIR :—Any of the concrete employed in the construction of the pipes mentioned.

MR. TINGLE :—I think you will find that that is given in the Appendix, to a certain extent, at any rate.

MR. SINCLAIR :—Thank you. Coming to the question of the reservoir wall cracked on the outside, it is interesting to see how the difficulty was got over ; but that method would have been unpracticable with a long wall. In a long wall of mass concrete, if such cracks did occur, the method which the author refers to would be of no avail. As a rule the crack goes right through, and whatever was done in the way of cutting the crack out and pointing up would be of no use the next time the temperature varied. I think other remarks I had in my mind have been expressed by the various speakers.

MR. TINGLE :—The President referred to the statement that concrete should be machine-mixed. Of course, I mean by that mixed in batches and not continuously. He also referred to a statement on page 198, where I say that in some instances artificial means are adopted to hasten the setting of the concrete while the tube is in the mould by exposure to hot air, with a view to releasing the moulds more quickly. Quite inadvertently he thought the statement referred to the Bonna pipe. It does not refer to any pipe at all, but is a general statement that in some instances concrete pipes are put under artificial means of drying in order to free the moulds. I do not say who does it and who does not do it, but I should very much like this Institute to ascertain what the real effect of such treatment is on concrete tubes. I think the question which the President asked with regard to the cruciform reinforcement in the Bonna pipes has been fully answered by Mr. Graham. With regard to the area of the failure in the reservoir first described, it really failed altogether. The walls gave out and the arch fell in. The area of the floor that was covered by furnace slag would be, as far as I can remember, about 20 ft. by 20 ft. The pothole under the concrete forming the floor was about 16 ft. diameter. And it was really over that pothole, and not over the whole of the floor. Neither was the concrete at all affected in other parts of the reservoir floor, but only in the portion where it was covered with this mixture of cement and furnace slag, or whatever it was. At any rate, it contained a consider-

able portion of sulphur. Mr. Moss-Flower suggested that cast-iron tubing would have been cheaper than the construction that was put in for the ejector chamber. As a matter of fact, I do not think that would have been the case. We had estimates for cast-iron tubing, and we chose the method that we adopted. Then he asked how soon did the tide follow on the laying of the concrete. At the sea outfall we used to work until we were driven out by the water—that is to say, as soon as the water began to get up to the level of the sideboards we closed the ends and the top boards, and the tide water would then come on immediately. Perhaps we could not go to that place again to continue the work for some time—two months or even three months—before we could get a quiet set of tides. We could only work just in the spring tides. Professor Dixon asked how the joints were made in the pipes at Norwich and Swansea. On page 197 you will see a section (Fig. 10) taken through the pipe and collar. The collar, reinforced with both spirals and longitudinals, is slipped over, the whole thing is filled in with cement grouting and also bitumen close to the joints, as shown in the figure; but if you look on the table below me here you will see an actual pipe, showing the method of the collar and the actual reinforcement. A Bonna pipe is lying there, and I think you will see quite clearly how the pressure is provided for at the joints. You may take it from me that the joint is one of the strongest parts of the pipe, if not the strongest, with regard to resisting internal pressure. There is no percolation if the pipe is properly made. Mr. Butler to a certain extent has somewhat misunderstood or has not quite realized what I meant. Perhaps it was my fault in not making it sufficiently clear in the paper, but I referred to those old papers as showing what was done in those old days in concrete and in concrete pipe-making and the difficulties that the men who took that initial work up had to deal with, and I gave an instance of that with regard to the quality of the cement that they were getting. I am sure that in 1875 and that period, unless you were very careful indeed, you would not get a cement that was very finely ground, and, in fact, Mr. Butler

—not the Mr. Butler who is here, of course, but the late Mr. J. W. Butler, of East Greenwich—was content if he could get a cement that would pass a 50 linear gauge. That is most extraordinary, because of course now we all know we can get cement that will only leave 5 per cent. on 180; but I do not think—at least, I hope—that no one else will fall into the pit which I seem to have laid for Mr. Butler, and think that these statements with regard to fineness and tests and all the rest of it are dealing with modern work. They are not, but there are instances given of modern cements in the Appendix: the British Standard, the 180, and all that sort of thing, which are modern cements; and, of course, I quite agree that you can get a better cement in England now if you specify it than you can get anywhere abroad. That is my view, and it is my experience. I have never found any foreign cement as good as the best British. Certainly in the south-east of Europe it is not, and there we always used to try to get a British cement. Of course, the tests referred to in Mr. J. W. Butler's paper are no good at all as illustrations of present-day practice. I want to make that quite clear. Mr. Butler asked why furnace slag was put in, but I am sorry to say that I do not know. I found it there when I was called in. I should say that the pothole had probably not been drained when they put in the floor, and it probably washed out some of the cement and left a porous concrete. Then they put this on the top in the hope of stopping it. That is my idea, but I really do not know. Mr. Butler referred to the specification of the cement in the Seaford outfall. That is only an extract from the specification, and he is therefore quite right. It is a curious thing that he should have queried it, because he was the gentleman who tested the cement to our specification, and I got a very severe specification too. Therefore if he looks up his records he can get all the information he requires. Of course, Mr. Sharp pointed out that the experiments on page 187 were to show the figures and the results that they did get in those days. Mr. Bylander asked a question as to the collars taking up the pressure, and the same remark applies to that as to Professor Dixon.

The collars are shown in Fig. 10, and can be seen in the paper.

MR. BYLANDER :—The longitudinal force is taken up by shear from the cement grouting in the joint?

MR. TINGLE :—That is so to a certain extent, but, you see, you get a plate of metal in the collar and also bars which will take up some of the shear. With Mr. Sharman's remarks concerning the importance of the quality of the aggregate, and the condition of the aggregate, and the grading of the aggregate, I entirely agree. Mr. Graham spoke of two reservoir failures, and I do not think that that is quite the case. The second reservoir I described is a failure to the extent that during a very hot summer we got slight cracks in the wall that was exposed to the sun's rays, but I hardly myself consider that is a failure in the sense that any water was lost. There was no water came through at all. I simply illustrated it as showing that even here in England you might get a serious accident due to temperature stress, through one side of the reservoir wall being heated probably up to 130 or 140 degrees, when you get 100 degrees in the shade at Greenwich, as we had that summer, and the other side being in contact with water at a temperature from the chalk of about 55 or 56 degrees. There is thus a difference of about 100 degrees on the two faces of the reservoir wall, inside and outside; and I assure you that if that had been a long wall, as one gentleman remarked, it would probably have failed altogether. I also agree with Mr. Graham that the first reservoir I described did fail through the pressure of the arch. There is no doubt about that, the wall was not strong enough to stand the pressure of the arch.

FIFTY-FIFTH ORDINARY GENERAL MEETING

THURSDAY, JANUARY 21, 1915

THE FIFTY-FIFTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, January 21, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I. Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

The following were elected :—

MEMBERSHIP.

WALTER FRANCIS CHRISTMAS, Principal Designer and Estimating Engineer to Richard Moreland & Son, Ltd., 80 Goswell Road, E.C.

PERCY WILLIAMS, Engineer, Shell House, 25 Bishopsgate, E.C.

ASSOCIATE-MEMBERSHIP.

UMBERTO FRANCESCO PESENTI, Civil and Mechanical Engineer, c/o British Reinforced Concrete Engineering Co., Ltd., 82 Victoria Street, London S.W.

Studentship.

Frederick Arthur Sanders, Junior Draughtsman, c/o British Reinforced Concrete Engineering Co., Ltd., 82 Victoria Street, S.W.

Mr. Ewart S. Andrews, B.Sc., M.C.I., then read his paper as follows :—

SOME MODERN METHODS OF ARCH CALCULATION.

The arch is a form of structure which possesses great advantages from the standpoints of beauty and economy, and from the earliest times the arch has been used in all kinds of constructional work. In the present paper we shall not consider arches at all from the point of view of architectural styles or orders, but will restrict our consideration to the calculation of the stresses in them.

We shall find that the arch presents points of considerable difficulty from this point of view, and that the resulting formulæ are elaborate. Some of our members may contend that the formulæ are too elaborate, and that "simple practical rules" are just as good ; our answer to such contention is that we will welcome such simple rules if they really are as good. The difficulty is that unless simple rules are applicable over a wide range, and have been fully tested by scientific experiment, there is considerable danger in their use.

There is, unfortunately, among some practical engineers a strong antipathy to complicated formulæ—an antipathy which is usually stronger in proportion as the formulæ are not understood—but closer acquaintance with such formulæ, and some useful spadework in the form of the compilation of tables and diagrams, usually helps to dispel much of the dread. The primary reason of the difficulty in the determination of the stresses in arches arises from the fact that in most cases the arch is what is called a "statically indeterminate structure," so that the forces acting upon the arch cannot be found by the ordinary laws of statics. Exactly similar difficulties arise in the case of stiffened suspension bridges, continuous beams, and slabs.

Determination of Reactions of an Arch.

The stresses in an arch can be found as soon as we can find the magnitude and position of the reactions ; these reactions R may be considered as compounded of vertical components V and horizontal thrusts H ; if, as is usual, all the loads on the arch are vertical, the horizontal thrust H must be the same at each end.

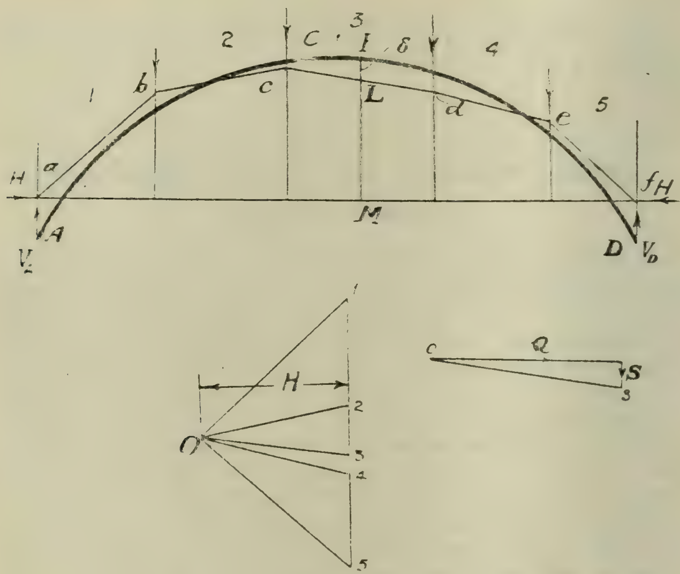


Fig. 1.

We can then draw the *line of pressure, linear arch, or equilibrium polygon* (three alternative names for the same thing), *a b c d e f*, Fig. 1, for the given load system upon the arch A C D.

Then by Eddy's theorem that " the bending moment at any point of an arch is equal to the product of the horizontal thrust into the vertical intercept between the centre line of the arch and the line of pressure," we have—

$$\text{Bending moment at any point } P = H\,\delta$$

If the line of pressure comes below the centre line of the arch, the upper surface or *extrados* of the arch will tend to become in tension ; and if the line of pressure is above the centre line of the arch, the lower surface or *intrados* of the arch tends to become in tension.

Stresses in the arch.—To obtain the stresses in the arch we first find the thrust or normal pressure Q and the shearing force S at the point by resolving the thrust O 3 at the point under consideration along and perpendicular to the centre line of the arch at the given point.

Then if A is the area of the section and M_c, M_t the compression and tension moduli, we have—

$$\text{Maximum tension stress} = t = \frac{H\,\delta}{M_t} - \frac{Q}{A} \quad . \quad (1)$$

$$\text{Maximum compression stress} = c = \frac{H\,\delta}{M_c} + \frac{Q}{A} \quad . \quad (2)$$

$$\text{Mean shear stress} = s = \frac{S}{A} \quad . \quad . \quad . \quad . \quad (3)$$

There is therefore no difficulty whatever in the calculation of stresses in arches when once we know the horizontal thrust.

Cases in which the horizontal thrust can be found without the elastic theory of arches.—These are as follows :—

1. Parabolic arch uniformly loaded over whole span—

$$H = \frac{w\,l^2}{8\,v}$$

w = load per unit length,
l = span of arch,
v = rise of arch.

2. Parabolic arch with uniform load extending from an abutment to the centre—

$$H = \frac{w\,l^2}{16 v}$$

3. Arch of any shape provided with three hinges. In this case the line of pressure can be drawn by the well-known

graphical construction for making a link-polygon pass through any three given points ; then the polar distance of the vector diagram gives the horizontal thrust desired.

Elastic Theory of Arches.

This theory, to which we will give our principal attention, is based upon a consideration of the deformations which an arch-rib obeying the ordinary laws of bending will receive. From these deformations we are able to calculate the horizontal thrusts.

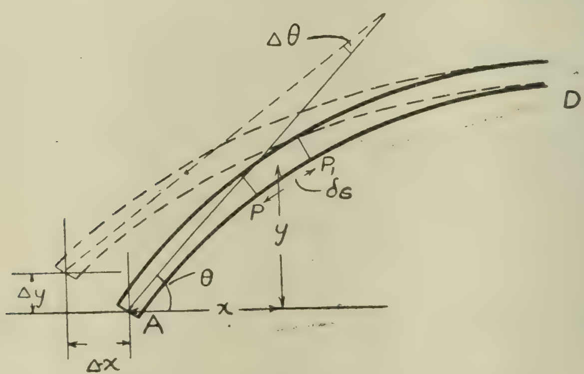


Fig. 2.

Let δs (Fig. 2) represent a very short length between two points P P_1 of the arch-rib A D, the end D of which is considered as fixed relatively to the other end A, and suppose that the bending moment along this very short length is B. Then the full lines indicate the position of the rib before bending and the dotted lines represent the position after bending.

If E is the elastic (Young's) modulus and I the moment of inertia of the rib along P P_1 ; it can be shown by the theory of beams that—

Δy = vertical displacement of A due to bending

$$= \sum_A^D \frac{B\,x\,\delta s}{E\,I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$\Delta x =$ horizontal displacement of A due to bending

$$= \sum_{A}^{D} \frac{B\,y\,\delta s}{E\,I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$\Delta \theta =$ angular change of tangent to rib at A

$$= \sum_{A}^{D} \frac{B\,\delta s}{E\,I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

These are the general formulæ upon which all the special formulæ are based, and it should be noted in passing that they are the deformations due to bending only and do not include those due to direct thrust ; we will refer to the effect of the latter at a later stage.

We will now apply these general formulæ to the most common special cases of rigid arches, viz.—

1. Arches with two hinges or pin joints, *i.e.* Two-pinned arches.
2. Arches without hinges, *i.e.* Fixed arches.

1. Two-pinned Arches.

If an arch is provided with hinges at the points A and D (Fig. 3)—practically always at the springings—there can be no bending moment at A or D, and if A and D are fixed, the horizontal displacement of D must be zero.

We thus have—

$$\Delta x = \sum_{A}^{D} \frac{B\,y\,\delta s}{E\,I} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

From Equation (4) we can calculate the value of the horizontal thrust as follows :—

Let B_o be the "free B.M." at any point P, *i.e.* the bending moment which would occur at P for the given load on a simply supported beam A D.

Then B.M. at $P = B =$ free B.M. minus moment of thrust H $= B_o - H \cdot y$,

$$\therefore \Sigma B y \delta s = \Sigma (B_o y - H y^2) \delta s$$

$$= \Sigma B_o y \delta s - \Sigma H y^2 \delta s$$

$$= \Sigma B_o y \delta s - H \Sigma y^2 \delta s \quad . \quad . \quad (5)$$

$$\therefore \sum_A^D \frac{B_o y \delta s}{E I} - H \sum_A^D \frac{y^2 \delta s}{E I} = 0$$

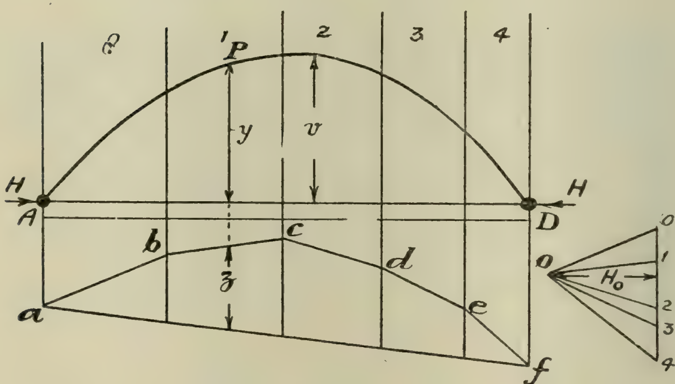


FIG. 3.

i.e.—

$$H = \frac{\sum_A^D \frac{B_o y \delta s}{E I}}{\sum_A^D \frac{y^2 \delta s}{E I}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

If E is constant, as it is in nearly every case—

$$H = \frac{\sum_A^D \frac{B_o y \delta s}{I}}{\sum_A^D \frac{y^2 \delta s}{I}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

If our free bending moments are obtained graphically we set down the loads on a vector line and draw a trial line of pressure with a thrust H_o, then—

$$B_o = H_o z$$

So that—

$$H = H_o \times \frac{\sum_A^D \frac{yz\,\delta s}{I}}{\sum_A^D \frac{y^2\,\delta s}{I}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

If the arch is of constant section, I is constant, and our equation becomes—

$$H = \frac{\Sigma_A^D B_o y\,\delta s}{\Sigma_A^D y^2\,\delta s} \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

For convenience, we may call $\Sigma y^2 \delta s$ the "arch-square sum" and $\Sigma B_o y \delta s$ the "load-arch sum," so that we have in words—

$$\text{Horizontal thrust} = \frac{\text{load-arch sum}}{\text{arch-square sum}}$$

By taking equal distances apart *along the arch*, *i.e.* equal values of δs, our equation becomes—

$$H = \frac{\Sigma_A^D B_o y}{\Sigma_A^D y^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

which can be easily found by tabulation.

The link polygon drawn with this thrust as a polar distance gives the true line of pressure required.

Arch varying in section.—If the arch varies in section, we may either divide each value in the sum by the corresponding value of I, or else take varying values for δs so that $\frac{\delta s}{I}$ is constant, thus getting as before the value given in Equation (11).

The line of pressure cannot be wholly above or below the centre

line.—This important fact can be deduced from either of the Equations (4) or (5); taking Equation (4), for instance, y, E, I and ∂s are always positive, so that B must sometimes be positive and sometimes negative if the summation is to be zero. This can only occur if the line of pressure cuts across the centre line of the arch.

Special Cases of Two-pinned Arches.

(*a*) **Parabolic Arches.**—In this case the summation in Equation (10) can easily be calculated mathematically for an isolated load if the section of the arch changes so that the moment of inertia varies as the secant of the angle of inclination of the arch to the horizontal.

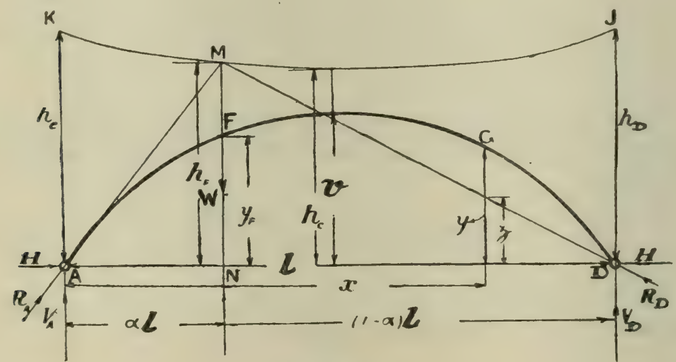


Fig. 4.

If an isolated load W be placed at a point F at distance αl (Fig. 4) from A, we get—

$$H = \frac{5}{8}\,\frac{W\,l\,\alpha\,(1-\alpha)\,(1+\alpha-\alpha^2)}{v} \quad . \quad . \quad . \quad (11)$$

Reaction Locus.—When dealing with isolated loads—and any load system can always be considered as divided up into isolated loads—one simple procedure for obtaining the reactions is by means of the "Reaction Locus." In practice, the load is often distributed to the arch by a number of

vertical columns, so that even with a uniformly distributed load on the span, the load on the arch itself is concentrated at a number of points.

The "Reaction Locus" is a line which gives the point of intersection of the two reactions for any position of an isolated load.

If, for instance, in Fig. 4 K M J is the reaction locus, then, if an isolated W be placed at the point F, and the vertical through F intersects the reaction locus at M, the lines M A, M D obtained by joining the point M to each of the hinges give the directions of the resultant reactions R_A and R_D at the hinges. This gives us the line of pressure A M D at once, and the bending moment at any point is equal to the product of the horizontal thrust H and the vertical distance between the line of pressure and the centre line of the arch at the point. If the reaction locus is known, and h_F is its height at the point F, then the value of the horizontal thrust H is readily calculated as follows :—

The Δ M A N must represent to some scale the Δ of forces at the point A.

$$\therefore \frac{V_A}{H} = \frac{MN}{NA} = \frac{h_F}{a\,l}$$

$$\therefore H = \frac{V_A\,a\,l}{h_F} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

and $V_A = W(1-a)$.

$$\therefore H = \frac{W\,a\,(1-a)\,l}{h_F} \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

At the point F the bending moment will be equal to $H \times MF$, *i.e.* B.M. at $F = H\,(h_F - y_F)$

$$= W\,a\,(1-a)\,l\left\{1 - \frac{y_F}{h_F}\right\} \quad . \quad . \quad . \quad . \quad (14)$$

For the parabolic arch this gives—

$$h_F = \frac{1{\cdot}6\,v}{1 + a - a^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

Equations (11) and (15) give the following values :—

a	h_F (Multiply by v)	H $\left(\text{Multiply by } \frac{W\,l}{v}\right)$	
0	1·60	0	1·0
0·1	1·47	0·061	0·9
0·2	1·38	0·116	0·8
0.3	1·32	0·159	0·7
0·4	1·29	0·186	6·6
0·5	1·28	0·195	0·5
	h_F	H	a

By dividing our loading up into isolated loads acting at these points we can tabulate the value of H for each load, and thus by adding together obtain the total thrust for the whole load system.

NUMERICAL EXAMPLE.—*Take the case of a two-hinged parabolic arch of* 120 *ft. span, and rise* 20 *ft. with a live load of* 20,000 *lb. per panel, and a dead load of* 10,000 *lb. per panel, the load being distributed at* 10 *points, as shown* (*Fig.* 5).

We will first tabulate values of H at each point per unit load ; by symmetry we need only consider one-half of the span.

We then get—

Point.	y (ft.).	h (ft.).	H (for unit load).
1	7·2	29·4	0·368
2	12·8	27·6	0·696
3	16·8	26·4	0·953
4	19·2	25·8	1·116
5	20	25·6	1·172

Now take the live load on the left-hand half of the span, and the dead load over the whole span, and find the thrust for the load at each point. We then get—

Point.	Load (in ten thousands of pounds).	H (in ten thousands of pounds).
1	3	$3 \times 0{\cdot}368 = 1{\cdot}104$
2	3	$3 \times 0{\cdot}696 = 2{\cdot}088$
3	3	$3 \times 0{\cdot}953 = 2{\cdot}859$
4	3	$3 \times 1{\cdot}116 = 3{\cdot}348$
5	2	$2 \times 1{\cdot}172 = 2{\cdot}344$
6	1	$= 1{\cdot}116$
7	1	$= 0{\cdot}953$
8	1	$= 0{\cdot}696$
9	1	$= 0{\cdot}368$
	Total ...	$= 14{\cdot}878$

Now calculate the B.M. at $\frac{1}{4}$ and $\frac{3}{4}$ spans.

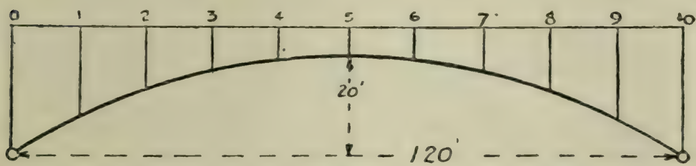


FIG. 5.

The vertical reaction at the left-hand end

$$= \frac{10 \times 10000}{2} + \frac{5 \times 20000 \times 3}{4} - 15{,}000 = 110{,}000 \text{ lb.}$$

∴ B.M. due to load at $\frac{1}{4}$ span

$$= 110{,}000 \times 30 - 30{,}000\,(18 + 6) = 2{,}580{,}000 \text{ ft.-lb.}$$

At $\frac{3}{4}$ span working from other end where reaction = 70,000.

∴ B M. due to load

$$= 70{,}000 \times 30 - 10{,}000\,(18 + 6) = 1{,}860{,}000 \text{ ft.-lb.}$$

At $\frac{1}{4}$ and $\frac{3}{4}$ span $y = 15$ ft.

∴ B.M. due to thrust $= 148{,}780 \times 15 = 2{,}231{,}700$ ft.-lb.

Positive B.M. $= 2{,}580{,}000 - 2{,}231{,}700 = 348{,}300$ ft.-lb.

Negative B.M. $= 2{,}231{,}700 - 1{,}860{,}060 = 371{,}700$ ft.-lb.

(*b*) **Circular Arches.**—In this case, referring to Fig. 6, it can be shown that for an isolated load W

$$H = \frac{W[\sin^2\theta - \sin^2\phi - 2\cos\theta(\cos\theta - \cos\phi + \theta\sin\theta - \phi\sin\phi)}{(4\,\theta\cos^2\theta + 2\,\theta - 3\sin 2\,\theta)} \quad (16)$$

Semicircular arch.—Here $\theta = 90° = \frac{\pi}{2}$. This gives—

$$H = \frac{W\cos^2\phi}{\pi} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

For load W at the crown, where $\phi = 0$, this gives—

$$H = \frac{W}{\pi} = 0{\cdot}318\ W \text{ nearly} \quad . \quad . \quad . \quad . \quad (18)$$

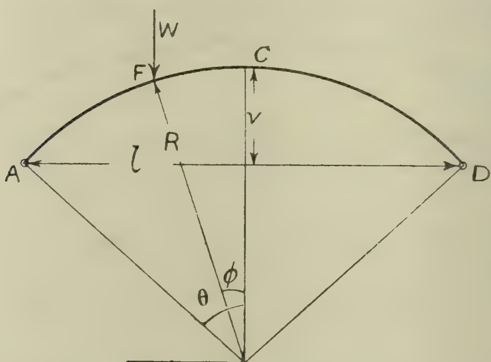


FIG. 6.

From the table on p. 264 for a parabolic arch, we get for the crown $H = \frac{0{\cdot}195\ W\,l}{v}$, and for $v = \frac{l}{2}$, which will give the same height of crown as the semicircular arch, $H = 0{\cdot}390\ W$.

90° *Circular arch.*—If the angle subtended at the centre of a circular arch is 90°, $R = \frac{1}{\sqrt{2}}$ and the rise v will be equal to

$$\frac{l}{\sqrt{2}} - \frac{l}{2} = 0{\cdot}207\ l.$$

This is nearly one-fifth of the span, and is a rise fairly common in practice.

In this case $\theta = 45° = \frac{\pi}{4}$.

The equation then gives the following values :—

Angle ϕ.	Value of H in terms of W.	Angle ϕ.	Value of H in terms of W.
0°	0·909	$33\frac{3}{4}$	0·315
$11\frac{1}{4}$°	0·835	45°	0·000
$22\frac{1}{2}$°	0·607		

60° *Circular arch.*—If the angle subtended at the centre is 60° the rise will be equal to $l\left(1 - \frac{\sqrt{3}}{2}\right) = 0 \cdot 134\, l$. This is between $\frac{1}{7}$ and $\frac{1}{8}$ of the span.

In this case $\theta = 30°$ and $d + R = l$, and the following results are obtained :—

Angle ϕ.	Value of H in terms of W.	Angle ϕ.	Value of H in terms of W.
0°	1·44	$22\frac{1}{2}$°	0·53
$7\frac{1}{2}$°	1·31	30°	0·00
15°	1·00		

These values are plotted in Fig. 7 upon the same base as for the 90° arch.

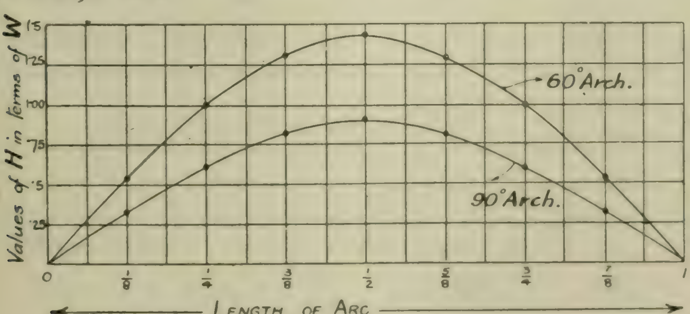


FIG. 7.

Reaction loci.—Fig. 8 shows the reaction locus for a semi-circular arch ; the other cases give the following values from which the loci can be drawn.

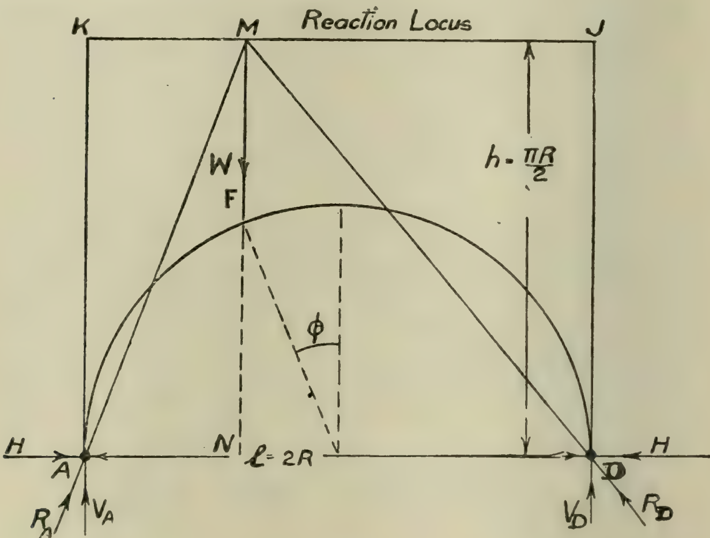


Fig. 8.

90° *Arch.*

Angle ϕ.	h.	Angle ϕ.	h.
0°	0·276 L	22½°	0·291 L
11¼°	0·277 L	33¾°	0·303 L

60° *Arch.*

Angle ϕ.	h.	Angle ϕ.	h.
0°	0·174 L	15°	0·183 L
7½°	0·178 L	22½°	0·188 L

Temperature Thrusts in Two-pinned Arches.

Let t° be the temperature range and β the coefficient of linear expansion; then if free to expand the span would become $l\,(1 + \beta\, t^\circ)$, *i.e.* $\Delta\, x = l\, \beta\, t^\circ$. The thrust H_T induced must be such as to bring this back to zero and $B = H_T \,.\, y$; *i.e.*—

$$\sum_A^D \frac{B\, y\, \delta\, s}{E\, I} = l\, \beta\, t^\circ$$

i.e.—

$$H_T \sum_A^D \frac{y^2\, \delta\, s}{E\, I} = l\, \beta\, t^\circ$$

$$H_T = \frac{E\, I\, l\, \beta\, t^\circ}{\sum_A^B y^2\, \delta\, s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

$$= \frac{E\, I\, l\, \beta\, t^\circ}{\text{arch-square sum}}$$

Parabolic arch.—(I = moment of inertia at crown)—

$$\text{arch square sum} = \frac{8\, v^2\, l}{15}$$

$$\therefore\ H_T = \frac{15\, E\, I\, \beta\, t^\circ}{8\, v^2}$$

Taking for steel $\beta = 6 \cdot 7 \times 10^{-6}$ per ° F. and $E = 30 \times 10^6$ lb. per square inch, this gives—

$$H_T \text{ for steel} = \frac{375\, I\, t^\circ}{v^2}$$

Taking for concrete $\beta = 6 \cdot 0 \times 10^{-6}$ per ° F and $E = 2 \times 10^6$ lb. per square inch—

$$H_T \text{ for concrete} = \frac{22 \cdot 5\, I\, t^\circ}{v^2}$$

Circular arch.—Semicircular arch—

$$H_T = \frac{16\,E\,I\,\beta\,t^o}{\pi\,l^2}$$

$$= \frac{1023\,I\,t^o}{l^2} \text{ for steel}$$

$$= \frac{61 \cdot 1\,I\,t^o}{l^2} \text{ for concrete}$$

90° arch—

$$H_T = \frac{E\,I\,\beta\,t^o}{0 \cdot 02503}$$

$$= \frac{8040\,I\,t^o}{l^2} \text{ for steel}$$

$$= \frac{480\,I\,t^o}{l^2} \text{ for concrete}$$

60° arch—

$$H_T = \frac{E\,I\,\beta\,t^o}{0 \cdot 00996\,l^2}$$

$$= \frac{20200\,I\,t^o}{l^2} \text{ for steel}$$

$$= \frac{1200\,I\,t^o}{l^2} \text{ for concrete}$$

EFFECT OF AXIAL THRUST ON FORMULÆ.—As we have indicated on p. 259 the above formulæ apply only to bending deformations. A correcting factor may be taken as $\left(1 + \frac{15 g^2}{8 v^2}\right)$ where g is the radius of gyration of the section. To apply this correcting factor we divide by it the thrusts obtained as explained above; it will be appreciable only for flat arches.

TWO-PINNED ARCHES WITH TIE-RODS.

If a two-pinned arch has a tie-rod in place of immovable abutments, the horizontal displacement of one end relatively

to the other will be equal to the extension of the tie-rod caused by the force H.

$$\text{This extension} = \frac{\mathrm{H}\,l}{\mathrm{E\,A}}$$

where E = Young's modulus and A is the area of the tie-rod.

Therefore we get from (2), p. 5—

$$\frac{\mathrm{H}\,l}{\mathrm{E\,A}} = \sum_{\mathrm{A}}^{\mathrm{D}} \frac{\mathrm{B}\,y\,\delta s}{\mathrm{E\,I}}$$

$$= \sum_{\mathrm{A}}^{\mathrm{D}} \frac{\mathrm{B_o}\,y\,\delta s}{\mathrm{E\,I}} - \mathrm{H} \sum_{\mathrm{A}}^{\mathrm{D}} \frac{y^2\,\delta s}{\mathrm{E\,I}}$$

$$\therefore\ \mathrm{H}\left(\frac{l}{\mathrm{E\,A}} + \sum_{\mathrm{A}}^{\mathrm{D}} \frac{y^2\,\delta s}{\mathrm{E\,I}}\right) = \sum_{\mathrm{A}}^{\mathrm{D}} \frac{\mathrm{B_o}\,y\,\delta s}{\mathrm{E\,I}}$$

$$\mathrm{H} = \frac{\sum_{\mathrm{A}}^{\mathrm{D}} \frac{\mathrm{B_o}\,y\,\delta s}{\mathrm{E\,I}}}{\sum_{\mathrm{A}}^{\mathrm{D}} \frac{y^2\,\delta s}{\mathrm{E\,I}} + \frac{l}{\mathrm{E\,A}}} = \frac{\Sigma_{\mathrm{A}}^{\mathrm{D}} \mathrm{B_o}\,y\,\delta s}{\Sigma_{\mathrm{A}}^{\mathrm{D}} y^2\,\delta s + \frac{l\,\mathrm{I}}{\mathrm{A}}} \quad \text{if I is constant}$$

$$= \frac{\text{load-arch sum}}{\text{arch-square sum} + l\,g^2}$$

where g is the radius of gyration of the arch section.

Hingeless or Fixed Arches.

If the ends of an arch are securely fixed there can be no angular change between the tangents at the two ends.

$$\therefore\ \Delta\,\theta = \text{angular change from A to D} = 0$$

i.e.—

$$\sum_{\mathrm{A}}^{\mathrm{D}} \frac{\mathrm{B}\,\delta s}{\mathrm{E\,I}} = 0 \quad \text{. . . . . . . (19)}$$

In addition to this relation we have the rule that the vertical and horizontal deflections at A must be zero.

Our three equations for the fixed arch therefore are—

$$\sum_{A}^{D} \frac{B\,\delta s}{E\,I} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$\sum_{A}^{D} \frac{B\,y\,\delta s}{E\,I} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$\sum_{A}^{D} \frac{B\,x\,\delta s}{E\,I} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

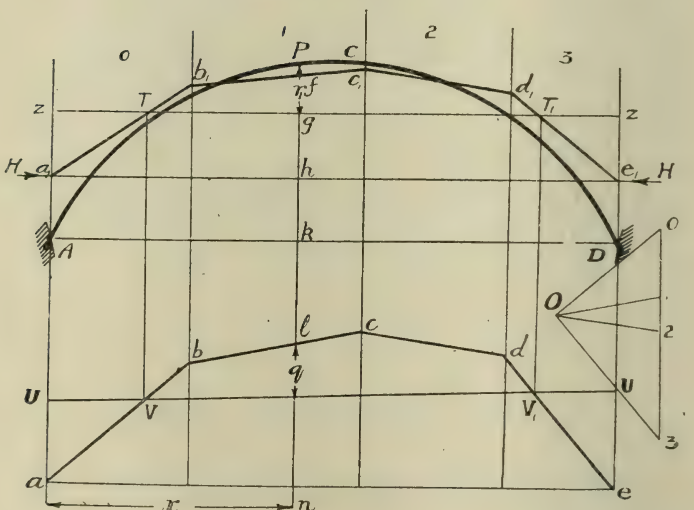


FIG. 9.

We no longer have the useful fact that the bending moments must be zero at the ends, and have therefore, in addition to the difficulty of the arch action, that of the fixity of the ends.

We shall see later that there is some similarity between the fixed arch and the fixed beam, and the treatment of both cases presents some difficulty.

Similar formulæ to those in the previous case can be obtained with reference to *reduced base lines* instead of the ordinary base as follows.

Suppose that the arch is divided up into a convenient number of elemental lengths such that $\frac{\delta s}{I}$ is constant; remembering that δs is measured along the arch and not along the span. (To save confusion on the drawing (Fig. 9) these divisions are not shown.) Let verticals be drawn through the centroids of these elemental lengths, and let P n be one of such verticals.

$a\ b\ c\ d\ e$ represents the free B.M. diagram drawn with any trial pole O and trial thrust H_o. Now let U U be the *reduced base line*, and be such that $\Sigma q = 0$ and also $\Sigma q x = 0$.

Also let a similar line $Z Z$ be drawn for the arch such that $\Sigma r_1 = 0$ and $\Sigma r_1 x = 0$. In most cases the arch is symmetrical, so that $Z Z$ is horizontal and is determined by the first relation only, *i.e.* $\Sigma r_1 = 0$. The line U U is the same as the base line for an ordinary fixed beam, with the exception that in the beam we take equal lengths along the span instead of along the arc.

Let U U intersect the free B.M. curve in V, V_1 which are projected vertically to cut $Z Z$ in T, T_1; the line of pressure can therefore be drawn through T T_1 as soon as the horizontal thrust H has been found.

We then have—

$$B = H\,(P\,h - f\,h)$$

$$= H\,(P\,g - f\,g)$$

$$= H\,.\,r_1 - H_o\,.\,q$$

(because $H\,.\,fg = H_o\,q$)

$$\therefore \sum \frac{B\,\delta s}{E\,I} = \frac{H\,\delta s}{E\,I}\Sigma r_1 - \frac{H_o\,\delta s}{E\,I}\Sigma q = 0$$

because $\Sigma r_1 = 0$ and $\Sigma q = 0$.

Therefore Equation (19) is satisfied. Also—

$$\sum_A^D \frac{B\,x\,\delta s}{E\,I} = \frac{H\,\delta s}{E\,I}\sum_A^D r_1\,x - \frac{H_o\,\delta s}{E\,I}\sum_A^D q\,x = 0$$

therefore Equation (5) is also satisfied because $\Sigma_A^D r_1 x = 0$ and $\Sigma_A^D q x = 0$. We are left with Equation (4)—

$$\sum_A^D \frac{B y \delta s}{E I} = 0$$

i.e.—

$$\frac{\delta s}{E I}\{\Sigma_A^D (H r_1 - H_0 q) y\} = 0$$

i.e.—

$$H \Sigma_A^D r_1 y - H_0 \Sigma_A^D q y = 0$$

or—

$$H = \frac{H_0 \Sigma_A^D q y}{\Sigma_A^D r_1 y} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

$$r_1 y = (r_1 + g k) r_1 = r_1^2 + g k . r_1$$

$$\therefore \Sigma^D r_1 y = \Sigma_A^D r_1^2 + g k \Sigma_A^D r_1 = \Sigma_A^D r_1^2 + g k . 0 = \Sigma_A^D r_1^2$$

Similarly—

$$q y = (r_1 + g k) q = r_1 q + g k . q$$

$$\therefore \Sigma_A^D q y = \Sigma_A^D r_1 q + g k \Sigma_A^D q$$

$$= \Sigma_A^D r_1 q + 0 = \Sigma_A^D r_1 q$$

$$\therefore H = H_0 \frac{\Sigma_A^D r_1 q}{\Sigma_A^D r_1^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

$$= H_0 \times \frac{\text{reduced load-arch sum}}{\text{reduced arch-square sum}}$$

The corrective term to allow for the effect of axial thrust in this case is $\left(\frac{1}{1+\frac{45\,g^2}{4\,v^2}}\right)$, *i.e.* H as above found is somewhat too high and has to be multiplied by the correcting factor for a rectangular section of depth d, $g^2=\frac{d^2}{12}$, therefore $\frac{45\,g^2}{4\,v^2}=\frac{15\,d^2}{16\,v^2}$. For flat arches this will come appreciable.

Summations for Reduced Base Lines.

Suppose that the number of elements is N, then—

$$k\,g=\frac{\Sigma\,y}{\mathrm{N}}$$

If the loading is symmetrical—

$$a\,\mathrm{U}=e\,\mathrm{U}=\frac{\Sigma\,l\,n}{\mathrm{N}}$$

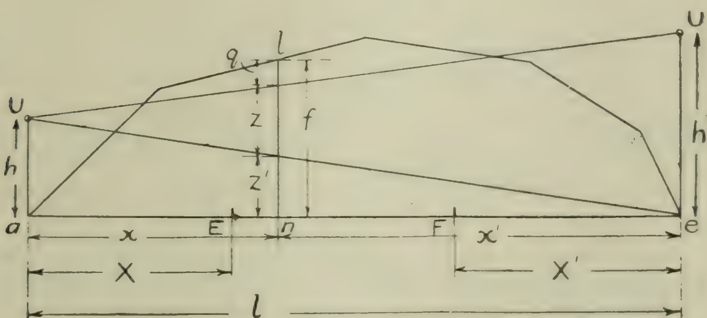


Fig. 10.

If the loading is not symmetrical we have to carry out a rather troublesome summation as follows: Join U e (Fig. 10); then $l\,n=q+z+z'$, *i.e.* $q=f-z-z'$.

Now—

$$z'=\frac{x'\,.\,h}{l}$$

and—

$$z=\frac{x\,h'}{l}$$

We have the relations $\Sigma\, q = 0$ and $\Sigma\, q\, x = 0$, *i.e.*—

$$\Sigma\, q = \Sigma\, f - \Sigma\, z' - \Sigma\, z$$

$$= \Sigma\, f - \sum \frac{h\, x'}{l} - \sum \frac{x\, h'}{l} = 0$$

i.e.—

$$\Sigma\, f = \sum \frac{h\, x'}{l} + \sum \frac{h'\, x}{l} \quad . \; . \; . \; . \; . \; . \; . \quad (22)$$

and—

$$\Sigma\, q\, x = \Sigma\, f\, x - \sum \frac{h\, x'^2}{l} - \sum \frac{h'\, x^2}{l} = 0$$

i.e.—

$$\Sigma\, f x = \sum \frac{h\, x'^2}{l} + \sum \frac{h'\, x^2}{l} \quad . \; . \; . \; . \; . \; . \quad (23)$$

Now let $X \times \Sigma\, z = \Sigma\, z\, x$, *i.e.*—

$$X = \frac{\Sigma\, z\, x}{\Sigma\, z} = \frac{\frac{h'}{l}\, \Sigma\, x^2}{\frac{h'}{l}\, \Sigma\, x} = \frac{\Sigma\, x^2}{\Sigma\, x} \quad . \; . \; . \; . \; . \; . \quad (24)$$

and—

$$X' = \frac{\Sigma\, z'\, x'}{\Sigma\, z'} = \frac{\Sigma\, x'^2}{\Sigma\, x'} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (25)$$

Now treat the f's, z's and z''s as forces, and we see that our Equations (22) to (25) amount to the ordinary equations for finding the reactions at points E, F for a series of forces equal to the f's. Let these reactions be R_E, R_F.

Then taking moments round F—

$$R_E = \frac{\Sigma\, f\, (x' - X')}{E\, F}$$

but—

$$R_E = \Sigma\, z = \frac{h'\, \Sigma\, x}{l}$$

$$\therefore\; h' = \frac{l\, R_E}{\Sigma\, x} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (26)$$

Similarly—

$$R_F = \frac{\Sigma f(x - X)}{E F}$$

and—

$$h = \frac{l R_E}{\Sigma x'} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (27)$$

These quantities are not so troublesome to calculate as the mathematical notation suggests. We can tabulate as follows :—

Segment.	x	x^2	x'	x'^2	$x - X$	$x' - X'$	f	$f(x - X)$	$f(x' - X')$
1									
2									
etc.									
	Σx	Σx^2	$\Sigma x'$	$\Sigma x'^2$				$\Sigma f(x - X)$	$\Sigma f(x - X')$

and thus readily find the values h and h' which determine the reduced base. The reduced ordinates q can then be calculated by the relation $q = f - h - \frac{x\, l\, (h' - h)}{l}$, and thus the summation for H can be effected.

Professor Cain's method.—Professor Cain * proceeds in a somewhat different manner to obtain the reduced base, his method being applicable only to the case of a symmetrical arch.

In this method we first draw the trial link polygon *a e* (Fig. 11), so that it is horizontal at the centre line C C of the span ; this is done by suitable choice of the trial pole.

Then if R R is the horizontal line tangential to the top of this link polygon through C, and N is the number of sections of the span considered, calculate—

$$k = \frac{\Sigma u}{N} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

and draw S S horizontal.

* "Theory of Solid and Braced Elastic Arches" (Van Nostrand).

Then calculate—

$$m = \frac{\Sigma\, u\, z}{\Sigma\, z^2} \cdot \frac{l}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

and set S U down and up respectively each equal to m, then U U is the reduced base required. Professor Cain then proceeds by summations which are similar to but, in our opinion, rather longer than those in our treatment.

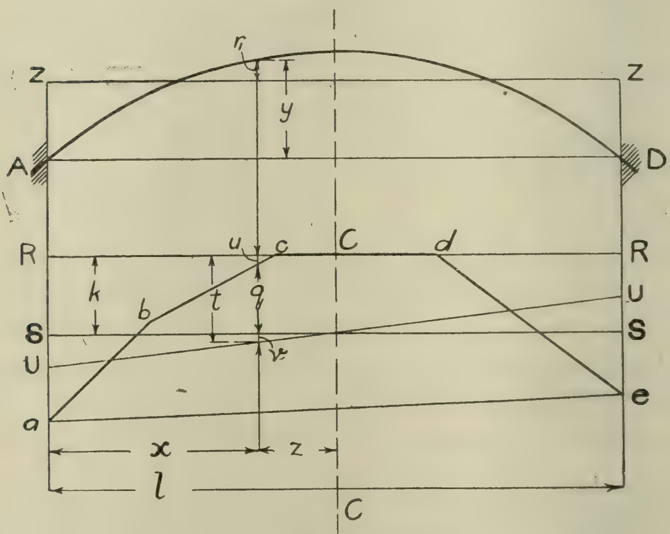


FIG. 11.

We will now prove that the above construction gives the same result as ours.

Our conditions are—

$$\Sigma\, q = 0$$

$$\Sigma\, q\, x = 0$$

Now—

$$q = t - u$$

$$\therefore\ \Sigma\, q = \Sigma\, t - \Sigma\, u$$

Now Σt is clearly equal to Σu, because for each ordinate on the left of C C is a corresponding one on the right as much less than a as t is greater than u, *i.e.*—

$$\Sigma q = 0$$

Also—

$$\Sigma q x = \Sigma t x - \Sigma u x \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

$$\Sigma t x = \Sigma k x + \Sigma v x$$

$$= k \Sigma x + \Sigma v \left(\frac{l}{2} - z\right)$$

$$= k \Sigma x + \frac{l}{2} \Sigma v - \Sigma v z$$

$$= k \Sigma x + \frac{l}{2} . 0 - \Sigma v z$$

$$= k \Sigma x - \frac{m}{\frac{l}{2}} . \Sigma z^2$$

because—

$$v = \frac{S U . z}{\frac{l}{2}} = \frac{m z}{\frac{l}{2}}$$

$$= k \Sigma x - \frac{2 m}{l} \Sigma z^2$$

$$\Sigma u x = \Sigma u \left(\frac{l}{2} - z\right)$$

$$= \frac{l}{2} \Sigma u - \Sigma u z$$

$$\therefore \Sigma q x = k \Sigma x - \frac{2 m}{l} \Sigma z^2 - \frac{l}{2} \Sigma u + \Sigma u z$$

$$k \Sigma x = k \left(\sum \frac{l}{2} - z\right) = \frac{k N l}{2} - k \Sigma z$$

and $\Sigma z = 0$ because the z's are symmetrical about C C and $\frac{l}{2}\Sigma u = \frac{k N l}{2}$ (from 28).

$$\therefore \Sigma q x = \frac{k N l}{2} - \frac{2 m}{l} \Sigma z^2 - \frac{k N l}{2} + \Sigma u z$$

$$= \Sigma u z - \frac{2 m}{l} \Sigma z^2$$

If this $= 0$, $m = \frac{l \Sigma u z}{2 \Sigma z^2}$ in accordance with Equation (29).

A modification of this method is described by Mr. H. A. Sewell in the *Proc. Am. Soc. C.E.* for 1912.

This method can be used for single isolated loads, enabling H and the end bending moments B_A, B_D to be calculated for each load by the rule $B_A = h$; $B_D = h_1$.

Parabolic Fixed Arches.

If a fixed arch is parabolic and varies in the same manner as we outlined for the two-pinned arch, *i.e.* $\frac{d s}{I} = \sec \theta$, the following results are obtained for an isolated load W.*

$$V_A = W (1 - a)^2 (1 + 2 a) \quad . \quad . \quad . \quad . \quad (31)$$

$$V_D = W (3 - 2 a) a^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

$$H = \frac{15 W L}{4 v} (1 - a)^2 a^2 \quad . \quad . \quad . \quad . \quad . \quad (33)$$

$$y_A = \frac{(10 a - 4) v}{15 a} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

$$y_D = \frac{(6 - 10 a) v}{15 (1 - a)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

* Proofs of these formulæ are given in the author's "Further Problems in the Theory and Design of Structures" (Chapman & Hall, Ltd.).

These results can be tabulated in convenient form as follows :—

a	$\frac{V_A}{W}$	$\frac{V_C}{W}$	H (in terms of $\frac{WL}{r}$)	$\frac{y_A}{r}$	$\frac{y_D}{r}$	
0·00	1·000	0·000	0·000	$-\infty$	+0·400	1·00
0·05	0·993	0·007	0·008	−4·667	+0·386	0·95
0·10	0·972	0·028	0·030	−2·000	+0·370	0·90
0·20	0·896	0·104	0·096	−0·667	+0·333	0·80
0·25	0·844	0·156	0·132	−0·400	+0·311	0·75
0·30	0·784	0·216	0·165	−0·222	+0·286	0·70
0·40	0·648	0·352	0·216	−0·000	+0·222	0·60
0·50	0·500	0·500	0·234	+0·133	+0·133	0·50
	$\frac{V_D}{W}$	$\frac{V_A}{W}$	H	$\frac{y_D}{r}$	$\frac{y_A}{r}$	a

Fig. 12 is drawn for the case of $a = 0·25$.

By means of this table we can divide our load up into a number of isolated loads, and for each position calculate H, $B_A = H\,y_A$ and $B_D = H\,y_D$.

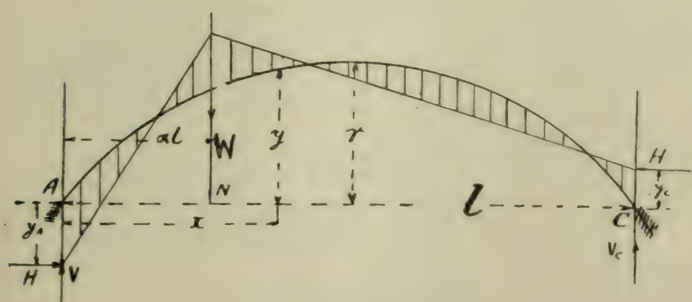


FIG. 12

Then the total thrust for all loads $= \Sigma\,H$ and total end B.M.'s for all loads $= \Sigma\,B_A$ and $\Sigma\,B_D$.

In the design of an arch other than parabolic it is a very good plan to make preliminary calculation on the assumption that the arch is parabolic, and then to make final check calculation by the more rigorous method when the dimensions have been decided upon.

Reaction Locus for Hingeless Parabolic Arch Rib.

We can obtain the reactions in the present case of the hingeless parabolic arch by a similar construction to that which we used for the arch with the two hinges; in the present case, however, we require two curves instead of one.

The intersection of the reactions will be along a straight line of height $h = 1{\cdot}2\,v$ (Fig. 13), and the reactions will be tangential to the line M N Q. This can be seen from Fig. 12, from which we get—

$$\frac{h - y_A}{a\,l} = \frac{V_A}{H} = \frac{W\,(1-a)^2\,(1+2\,a)}{\dfrac{15\,W\,l}{4\,v}\,(1-a)^2\,a^2} = \frac{4\,v\,(1+2\,a)}{15\,l\,a^2}$$

$$\therefore\ h - y_A = \frac{4\,v\,(1+2\,a)}{15\,a}$$

$$\therefore\ h = \frac{4\,v\,(1+2\,a)}{15\,a} + y_A$$

$$= \frac{4\,v\,(1+2\,a) + (10\,a - 4)\,v}{15\,a}$$

$$= \frac{v\,(4 + 8\,a + 10\,a - 4)}{15\,a}$$

$$= \frac{18\,a\,v}{15\,a} = 1{\cdot}2\,v$$

The line M N Q can be drawn by taking from the table on p. 281 the values of y_A and y_c for values of a from 0 to 0·5, then by drawing in the reactions at each point we shall be able to

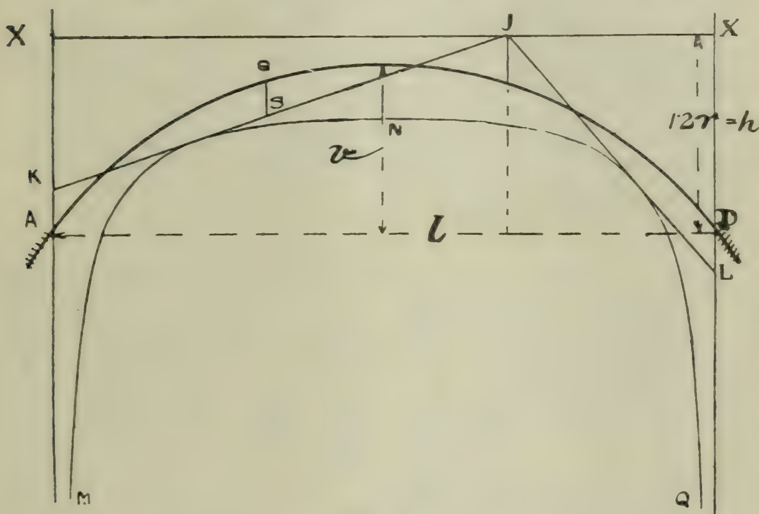


Fig. 13.

draw in the required curve, which is the "envelope" of the reactions. This curve is sometimes taken as two hyperbolas, the height at the centre being $\frac{2v}{3}$.

Temperature Thrusts for Fixed Arches.

In this case we get by similar reasoning to that in the case of the two-pinned arch—

$$H_T = \frac{E\,I\,l\,\beta\,t^o}{\text{reduced arch-square sum}}$$

and it acts along the line ZZ (Fig. 9).

In the case of the parabolic arch—

$$H_T = \frac{45}{4}\,\frac{E\,I\,\beta\,t^o}{v^2}$$

i.e. six times that in the corresponding case of two-pinned arches; it acts at a height $\frac{2v}{3}$ from the springings.

Lilly's Formulæ for Arches.

Professor Lilly, of Dublin, has derived some interesting formulæ* for arch design based upon a mathematical investigation of the equilibrium line upon similar lines to those adopted by Rankine and later by Alexander and Thompson.† The formulæ are for circular arches whose rise is not more than $\frac{1}{5}$ of the span.

The following approximate formulæ result—

$$w\,d = w_l$$

$$R = \frac{1 \cdot 66}{w}\sqrt{c\,w_l}$$

$$\frac{0 \cdot 182\,v}{R} = \frac{1}{2} + \frac{w_d}{w_l} + \frac{w_1}{w}$$

where R = radius of centre line in feet,
d = depth of arch ring in feet,
w_1 = weight of arch ring in lb. per cub. ft.,
w = weight in lb. per cub. ft. of load area above arch ring,
w_d = dead load in lb. per sq. ft.,
w_l = live load in lb. per sq. ft.;
c = safe compressive stress in material.

Conclusion.

In a paper of this kind it is manifestly impossible to deal fully with a subject upon which several long text-books have been written and it may very well be that many important points have been omitted. The author hopes, however, that a study of this paper will enable those members of this Institute, who have not had the opportunity of a complete study of the arch theory in its more general aspects, to follow the fundamental relations upon which scientific calculations are based and that their interest will be sufficiently aroused to encourage them to study the subject more fully.

* *Proc. Inst. C.E. Ireland*, 1914.
† "Elementary Applied Mechanics" (Macmillan).

DISCUSSION.

THE PRESIDENT (PROFESSOR HENRY ADAMS) :—I confess to being one of the practical engineers referred to who have a strong antipathy to complicated formulæ. I admit that to determine the true theoretical stresses in any given case is extremely useful as a study and of some value as keeping the practical and empirical formulæ within reasonable limits of accuracy, but I do not attach any importance to a calculation of the stresses in one part to the third decimal point and leaving the other parts

FIG. 1

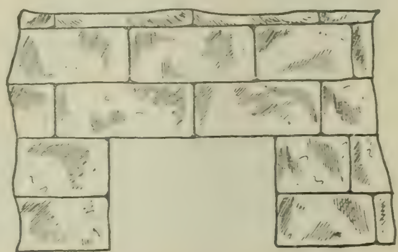


FIG 2

to take care of themselves in the units and the tens, which is, I think, too often the tendency of elaborate calculation. I am strongly in favour of graphic methods whenever they can be applied without running to any risk of appreciable error. It is astonishing how close one can get in graphic methods. Then, again, the author has told us that the arch has been used from the earliest times, but I have diagrams over there on the wall that show some earlier methods of carrying the superstructure over an opening without using an arch. In Fig. 1 you will see a very large stone put over the opening, carrying smaller stones above ; now we should call it a lintel. In Fig. 2 you will find indicated that by some cause or another the stone is broken through the centre, showing the next early architect who came along that he could use two cantilevers instead of the one beam

across; and then comes an ingenious man—he must have been an engineer, I think—who cuts back the two cantilevers and puts a bevelled stone voussoir in the centre, as in Fig. 3, and so is able to open the opening with much smaller stones. Then another

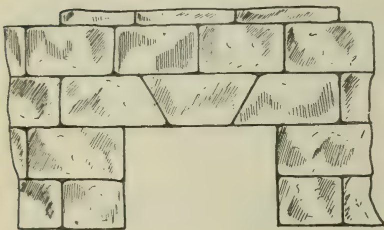


FIG. 3

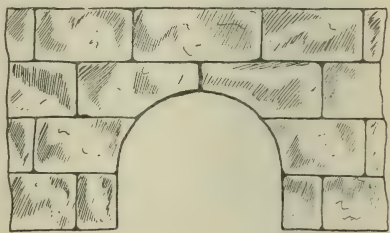


FIG. 4.

genius comes along and cuts away the soffiting so as to make a curved top, as in Fig. 4, not an arch, only shaped cantilevers. Of course you know the modern means of preventing the lintel from cracking, shown in Fig. 5, by the relieving arch turned over

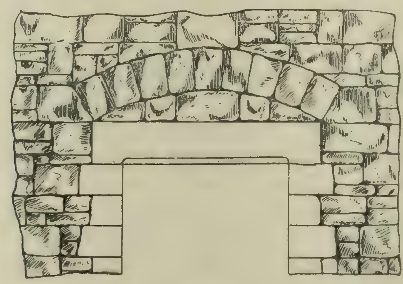


FIG. 5.

it. But I think the most interesting of all is the diagram, Fig. 6, which shows you an ordinary concrete lintel; but that contains within it an innumerable number of arches, from a deep arch of very small rise to a shallow arch of very great rise. The stresses resulting are tabulated below, from

CONCRETE LINTEL CONSIDERED AS AN ARCH.

Comparison of Concealed Arches.

Camber in inches.	Mean Depth, inches (d).	Mean Span, inches (l).	Measure of Thrust $t = \left(\frac{l}{d}\right)$	Depth of Arch, inches (D).	Comparative Stress per Unit of Area from Constant Load = $\left(\frac{t}{D}\right)$
1	1·06	37·21	34·77	11	3·16
2	2·18	38·14	17·46	10	1·74
3	3·25	38·87	11·96	9	1·33
4	4·37	39·26	8·98	8	1·12
5	5·52	39·48	7·15	7	1·02
6	6·63	39·46	5·95	6	0·99
7	7·68	39·29	5·12	5	1·02
8	8·71	38·88	4·46	4	1·12
9	9·59	38·32	3·99	3	1·33
10	10·49	37·58	3·58	2	1·79
11	11·32	36·74	3·24	1	3·24

which you will find that the one with the minimum stress, and therefore the concealed arch that you would look upon as the substitute for the lintel, is the one having half the depth for rise and the other half for arch. The angle of the skewback will vary in each case as shown by the end lines, and below will be found, in Fig. 7, the results tabulated in curves. These

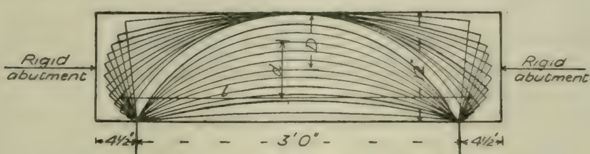


FIG. 6

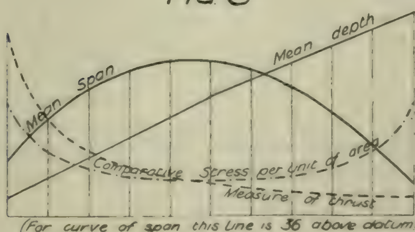


FIG. 7.

are some of my private collection of fifteen hundred lecture diagrams that I thought might be useful to you to-night. The concealed arch in the lintel reminds me of what has been said about some of the beautiful sculptures, that it was there beforehand, the sculptor only knocking away the waste material on the outside ! If you can imagine that, you have a true arch left inside the lintel, requiring only rigid abutments to make it fulfil its purpose as an arch.

As the concealed arch with rigid abutments would carry safely about four times the distributed load that the lintel with slack ends would do, it shows the importance of making a close, firm joint at the ends of every lintel.

MR. A. HEDLEY QUICK, Assoc.M.Inst.C.E., A.M.I.Mech.E., M.C.I. :—I wish to call attention to Equation 5 on page 260. The author there states that Δy, the vertical displacement of A, is represented by

$$\sum_{A}^{D} \frac{B\,x\,\delta s}{E\,I}$$

and this he equates to zero. This equation, I submit, is only true in the case of an arch which is fixed at one end. I think I could make myself clearer if I drew a diagram on the blackboard. When an arch is unequally loaded, a short element of the rib s has its slope and vertical position altered and takes up a new position, δs_1.

Take first the alteration in slope, as a consequence of which A receives both a horizontal and a vertical displacement, only the latter of which we need consider. The former is dealt with in Equation 4, and I have no comment to make thereon.

In Fig. 1 the triangles B A E and A A_1 C are similar, $\therefore A_1 C = \Delta y = \theta x$, where is expressed in circular measure. The total vertical displacements due to all the rib elements $= \Sigma \theta x$, which is another form of Equation 5, where a substitution has been made for θ in terms of B, E, and I.

Now, in Fig. 2, if the arch is hinged at A and D, the tangent to the rib at D moves through an angle θ_1, and point A receives in consequence a vertical

displacement $L\theta_1$, which is in addition to that due to the alteration in slope and must be added to it, $\therefore \Sigma\theta x + L\theta_1$ equals the whole vertical displacement of point A. But A does not move, $\therefore \Sigma\theta x + L\theta_1 = 0$. If,

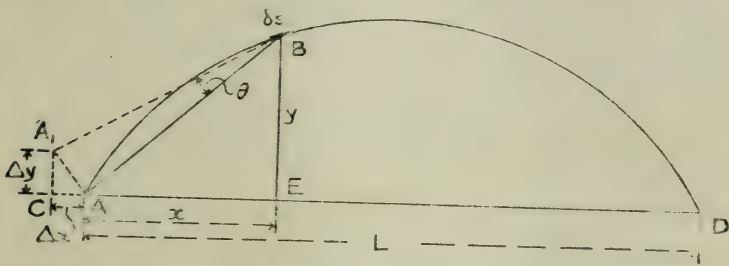


Fig 1.

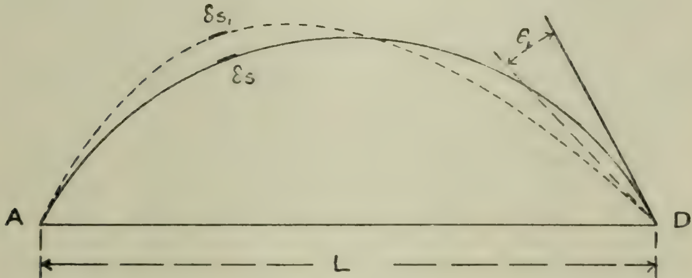


Fig 2.

however, the arch be rigidly fixed at D, $\theta_1 = 0$ and then $\Sigma\theta x = 0$. Really

$$\sum_A^D \frac{B x \delta s}{E I} = \text{the angular movement of } D \times A\,D \text{ in the case of two hinged arches.}$$

The angular movement of the rib at the hinges affects the vertical displacements only, and not the horizontal displacements. The only conditions that are necessary for determining the bending moments in a double-hinged arch are, first, the absence

of bending moments at the hinges, and second, the constancy of the span. This second condition is expressed by means of the integral in Equation 4.

On page 276, the second equation down, the substitutes for the summations of z and z' have been transposed. This, of course, does not affect the accuracy of the work, but it may confuse some people. It should read

$$=\Sigma f-\sum\frac{x\,h'}{l}-\sum\frac{x'\,h}{l}.$$

MR. W. CYRIL COCKING, M.C.I., next spoke.

MR. ALLAN GRAHAM, A.R.I.B.A., M.C.I. :—Mr. Andrews mentioned the fact that by designing an arch as a parabola it would not be necessary to settle the depth of the masonry on the concrete required, but this method could only apply to an open spandril arch when one can be certain that the deformation of the arch will take the form of a parabola. In the case of arches with heavy earthwork on top the line of pressure deviates considerably from the parabola, and for that reason I do not think that that formula would apply to such an arch. To design an arch of that type the real way is to set it out to please the architectural eye, and then to check it afterwards, as Mr. Cocking has suggested. In fact, my belief is that most of the designers of those large arch bridges draw them out to scale, as I have just suggested. After all, the experienced designer, if he uses his eye well, from his past experience can arrive at a very good approximation to the dimensions that are required; but that is not sufficient when the problem is a new one for them; we need the mathematician as well. We cannot erect large arches or bridges, except on previously erected models, without some leading mind furnishing us with the data from which to work. There is a general tendency to decry thinkers like Mr. Andrews when they attempt to get at root values for these formulæ; but no approximate formula can be arrived at successfully until we have really got the sound mathematical theory and basis from which to work. No amount of running

down the theorists will ever get over that fact. A practical man is entirely indebted to the theoretical man for almost all the knowledge he possesses. I am much indebted to Mr. Andrews for his paper, owing as I do a great deal to him for his written books on structural engineering, which, together with Professor Morley's, one can do a good deal. A good source of information on the subject of bridges is, I think, to be found in a good many papers written for the American Society of Civil Engineers, which give a lot of splendid data which can be applied by any one who is in the fortunate position of having to design some large bridge.

MR. E. FIANDER ETCHELLS, F.Phys.Soc., A.M.Inst.C.E. (Member of Council C.I.) :—We have very greatly enjoyed the gentle satire of the erudite teacher and his rebellious disciple. The paper itself is essentially one for the quietness of the study rather than for the clamour of the forum. The paper has dealt principally with the analytical aspect of the modern methods of calculation. When engineers are agreed as to the analysis we shall be able to proceed with the synthetic rules. We could then design the structure straight away. Having found the best synthetic rules, we shall be able to advance to the rule beloved by Mr. Cocking—"the boiled-down formula." At present there is still a great deal left to the judgment of the designer, and it is quite possible that, since the intricacy of Nature is far more complex than any formula, there will always be room for the judgment and experience of the designer or the contractor fighting against time. Those lay clients who have contracts to let don't usually give contracts for the most profound guess at philosophical truth. They usually give the contract to the one that quotes the cheapest price, and he is often the one that takes the most risk. There is no getting over that. We owe a debt of gratitude both to those who wish to discover the abstract truth and to those who wish to get a job done and get on with the next.

With regard to Eddy's theorem that "the bending moment at any point of an arch is equal to the product of the horizontal thrust into the vertical intercept

between the centre line of the arch and the line of presssure," I would like to say that this is merely one particular example out of the general theory of flexure. A few years ago some experiments were conducted by Mr. C. Batho in Canada on flexure under non-axial loading, and there the author came to the conclusion that our common rules for flexure required some amplification. I know that Mr. Andrews has read the paper to which I am referring, and I should like to ask him whether, in his recollection, the decision of the author was that our general mathematical theory of flexure is correct and exact, because if our general theory is correct Eddy's theorem follows from it and cannot be controverted.

THE PRESIDENT :—Before calling upon the author to reply I should like to put a practical case. (Here Professor Adams used the blackboard.) This is an arch, a half-brick, semi-circular arch, that was built as a cover only, having no load but its own weight to carry, 10 ft. span, and, of course, 5 ft. rise. As soon as the centering was taken away the arch fell. Can the author show us, shortly, how to find the direction of the reaction and the position of the horizontal thrust?

MR. EWART S. ANDREWS :—The paper is naturally of such an order that it does not readily admit of much discussion other than the outpourings of rebellious pupils, and I realized that when I wrote it. As a matter of fact, I purposely, I must admit, kept to rather mathematical lines, once having started on them, because I was very anxious that you should have a full record of the way in which these summations are made or could be made should you be willing to make them. That is what I am anxious about. I am not so short-sighted as to suggest that whenever you have to make a practical design you must immediately spend weeks on long calculations. That is the travesty of truth which the ultra-practical man always hurls at the theoretical man. And, of course, it is an attitude which seems to find a considerable amount of popular support. You will find, however, that if you go to the States, where they have made a very specialized study of construc-

tional engineering, they do think it worth while, even the practical men, to study the theoretical side of the subject. For instance, they recently had a paper on Columns, a most erudite paper, and the people who got up to discuss that paper showed by the discussion that they fully understood all the mathematical theories dealt with in the paper, and yet they were people such as the managers of this, that, and the other bridge company, and you find that the practical men there, in positions of commercial importance, are men who do understand the subject fully. They realize that the theoretical side has some value, and certainly they were familiar with it. People often refer to the Forth Bridge as being a kind of monument to the skill of the practical man who makes no calculations. People who talk like that are utterly ignorant of the attitude that Sir Benjamin Baker adopted. He was a man who paid the utmost regard to the questions of the purely mathematical and theoretical side of the subject. I know, for instance, that during a controversy a few years ago on the design of dams Sir Benjamin Baker, who was then in charge of the Assouan Dam, did not push the thing aside at all, but went into it absolutely thoroughly. Two of his assistants investigated it thoroughly, and at a meeting of the Institution of Civil Engineers, where the matter was discussed, he spoke very strongly in favour of all attempts of that kind as being of very great importance. Professor Adams says that he is one of those who believe in the practical way of doing a thing, and that for his part he thinks graphical methods are very valuable. I don't suppose there is anybody who has championed graphical methods more than I have, but I do not think Professor Adams can show me a graphical method of dealing with these arch problems. It is not that I advocate methods other than graphical, but you can only analyse a thing by analysis, and the graphical method is something which you use afterwards to obtain the results of your analysis. And then we had that interesting case of the arch lintel, the so-called hidden arch in the lintel, which is an extremely interesting thing ; but the difficulty there—I do not know how he made the calculations as to what the

thrusts are—is to say what is the arch, and when you have adopted what you will decide to regard as the arch, different methods of calculation will obtain different results. In regard to Professor Adams' problem on the board, I am very glad, if I may say so, that it did actually fall down; I should have expected it to fall.

I think Mr. Quick's objection to my statement is a valid one, and I think he could have proved it out of my own lips, as it were, because if you look up my "Theory of Structures" you will find that the vertical and horizontal shifts in that book are worked out at considerable length, and they are, I think, more correct there than in the paper.* Of course, it is not a very serious difficulty. The formula is correct for the diagram drawn, because we assumed it fixed. It is not, however, strictly correct, and I think the best thing will be to delete it.

Mr. Cocking spoke of the value of the simple rule, with which, of course, I entirely agree. If your simple rules are only what he calls boiled-down correct ones, I am in perfect agreement with him. My point is that you cannot expect the professors to do all the boiling down. You do occasionally hear a man saying: "What is the good of the professors doing all this mathematics? Why don't they prepare rules which any practical man can use in everyday work and so be useful to him in his profession?" In other words, Why does not the professor give us information so that we can take the fees while he does it for nothing? Of course, the great mistake that a large number of people make is as to the number of decimal points. There is no suggestion at all that you want to take a large number of decimal points in your calculations. Mr. Cocking spoke of the 4 per cent. error in beam sections which we must have on account of the rolling margin. That is perfectly true, and for that reason it is absurd to express results to a very large number of decimal points. At the same time, because you have a 4 per cent. error in your rolling margin, there is no reason why you should add a 20 per cent. error in your calculations, making a possible error of 24 per cent.

* They have now been corrected in the paper.—E. S. A.

We want to get our errors down as small as possible, and because we know there are some errors in some respects that is no reason why we should not eliminate all those errors which we can eliminate. To Mr. Graham I am very much indebted for the kind words which he said about me. I am, of course, to some extent championing the mathematical treatment of things, and that largely because I am so anxious that before people undervalue it they should understand it. That is the difficulty, that the most difficult criticism that you have to meet and to destroy is the criticism that comes from the man who does not understand what you have said, and although when you are going to have a small lintel to design you are not going to the trouble of making arch summations, which will take you two or three weeks perhaps, it does not follow that it is not worth your while to find out the way in which you should proceed when you wish to obtain greater accuracy than you can get by your ordinary methods. Mr. Etchells referred to a paper which he thought I could remember. I think he referred to a paper by Mr. Batho, read before the Canadian Society of Civil Engineers.

MR. E. FIANDER ETCHELLS :—Yes, that was it.

MR. ANDREWS :—I thank you, gentlemen, for the patience with which you have listened to this paper. It is a paper that, as Mr. Etchells said, is more to read than to listen to. It must be very monotonous to listen to me reading all those mathematical terms, but I do hope that you may find in it something that may be of value to you. The part which I think myself is the most valuable, or might be the most valuable, is from page 274 onwards, dealing with the fixed arch summations. I have found these very difficult to understand myself. The first method I have given you is one I have never seen published, but it is one I obtained from Professor Karl Pearson, to whom I am indebted for nearly all the knowledge I ever had on the subject.

The following is a mathematical solution of Professor Adams's problem, the assumption being that the ends are hinged.

Consider a short length of arc of the arch, the centre of which is at F, Fig. 6 (p. 266).

Then if it subtends an angle $d\,\phi$ at the centre and the arch weight w per unit length, we have weight of short length $= w\,\mathrm{R}\,d\,\phi$, therefore by Equation (17) (p. 266)—

$$\text{Thrust caused by element} = d\,\mathrm{H} = \frac{w\,\mathrm{R}\cos^2\phi\,d\,\phi}{\pi}$$

$$\text{Total thrust} = \mathrm{H} = \int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \frac{w\,\mathrm{R}\cos^2\phi\,d\,\phi}{\pi} = \frac{w\,\mathrm{R}}{\pi}\int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \cos^2\phi\,d\,\phi$$

$$= \frac{w\,\mathrm{R}}{\pi}\int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \left(\frac{1+\cos^2\phi}{2}\right) d\,\phi = \frac{w\,\mathrm{R}}{\pi}\left[\frac{\phi}{2} + \frac{\sin^2\phi}{4}\right]_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} = \frac{w\,\mathrm{R}}{2}$$

$$\text{Total weight of arch} = \mathrm{W} = w \times \pi\,\mathrm{R},$$

$$\therefore\ \mathrm{H} = \frac{\mathrm{W}}{2\,\pi} = \cdot 1592\,\mathrm{W} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The ordinate y of the line of pressure at F can be calculated as follows.

An elemental arc at Q contributes at F a bending moment equal to—

$$w\,\mathrm{R}\,d\theta \times \mathrm{N\,M} = w\,\mathrm{R}\,d\theta(\mathrm{R}\cos\theta - \mathrm{R}\cos a) = w\,\mathrm{R}^2(\cos\theta - \cos a)\,d\theta$$

$\therefore$ Total B.M. at F due to weight of arc from A to F

$$= \int_0^a w\,\mathrm{R}^2(\cos\theta - \cos a)\,d\,\theta$$

$$= w\,\mathrm{R}^2\left[\sin\theta - \theta\cos a\right]_0^a$$

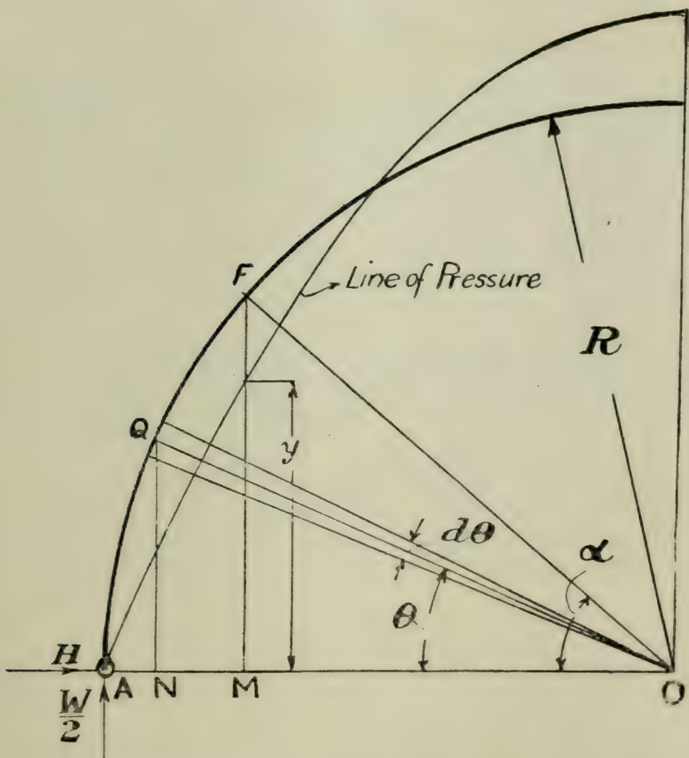
$$= w\,\mathrm{R}^2(\sin a - a\cos a)$$

$\therefore$ Free B.M. at $F = B_o$

$$= \frac{W}{2} \cdot A\,N - \text{Moment of weight of arc } A\,F \text{ about } F$$

$$= \frac{W\,R}{2}(1 - \cos\alpha) - w\,R^2(\sin\alpha - \alpha\cos\alpha)$$

$$= \frac{W\,R}{2}(1 - \cos\alpha) - \frac{W\,R}{\pi}(\sin\alpha - \alpha\cos\alpha) \quad (2)$$



Now—

$$y = \frac{B_o}{H} = \frac{B_o \cdot 2\pi}{W} \quad \text{(from 1)}$$

$$= R\{\pi(1 - \cos\alpha) - 2(\sin\alpha - \alpha\cos\alpha)\} \quad . \quad . \quad (3)$$

In using this formula α must be in radians. This gives the following values of $\frac{y}{R}$ for various values of α.

α in degrees	30	45	60	75	90
$\frac{y}{R}$	0·328	0·617	0·886	1·074	1·142

The actual B.M. at $F = B_o - H . PM$

$$= \frac{WR}{2}(1 - \cos\alpha) - \frac{WR}{\pi}(\sin\alpha - \alpha\cos\alpha) - \frac{WR\sin\alpha}{2\pi}$$

$$= WR\left[\frac{(1 - \cos\alpha)}{2} - \frac{(1{\cdot}5\sin\alpha - \alpha\cos\alpha)}{\pi}\right]$$

At the crown where $\alpha = 90°$ this gives—

$$B = \frac{WR}{2\pi}(\pi - 3) = 0{\cdot}0225\, WR$$

$$= \frac{WL}{89} \text{ nearly}$$

FIFTY-SIXTH ORDINARY GENERAL MEETING.

THURSDAY, FEBRUARY 4, 1915.

THE FIFTY-SIXTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, S.W., on Thursday, February 4, 1915, at 7.30 p.m.

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (President), in the Chair.

A Draft Specification for Reinforced Concrete Work was presented for discussion as a Joint Report of the Science and Reinforced Concrete Practice Standing Committees of the Institute.

MR. E. FIANDER ETCHELLS, A.M.Inst.C.E., F.Phys.Soc., etc., Chairman of the Science Standing Committee, who had acted as Chairman of the Joint Committee Meetings, introduced the Report, explaining its chief features.

MR. S. BYLANDER, Chairman of the Reinforced Concrete Practice Standing Committee, followed with further introductory remarks.

The following also contributed to the discussion: MR. RICHARD COULSON, F.S.I.; MR. GOWER PIMM, Assoc.M.Inst.C.E., M.C.I.; MR. MORGAN E. Yeatman, M.A., M.Inst.C.E., Member of Council, C.I.; MR. T. C. DAWSON, M.C.I.; MR. HENRY J. HARDING, M.C.I.; MR. E. P. WELLS, J.P., Past President, C.I.; MR. W. H. LASCELLES, M.C.I.;

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It being the intention to revise the Draft Specification in the light of the criticisms made in the Discussion, and it having been decided not to publish a Standard Specification for the time being, the report of the remarks made by the various speakers will not be printed.

INDEX

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I

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

(*FOUNDED* 1908. *INCORPORATED* 1909.)

TRANSACTIONS

AND

NOTES

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(a) To advance the knowledge of concrete and reinforced concrete, and other materials employed in structural engineering, and to direct attention to the uses to which these materials can be best applied.

(b) To afford the means of communication between persons engaged in the design, supervision and execution of structural engineering works (excluding all questions connected with wages and trade regulation).

(c) To arrange periodical meetings for the purpose of discussing practical and scientific questions bearing upon the application and use of concrete and reinforced concrete and other materials employed in structural engineering.

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CONTENTS

Discussion at the Meetings : Participation therein by—

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Donor.	*Literature.*
Institution of Civil Engineers ...	Bell on "The Lateral pressure and resistance of clay and the Supporting power of clay foundations."
	2nd Report of Committee on Reinforced Concrete.
Permanent International Association of Navigation Congresses	Minutes.
	Report of Executive Committee.
	Programme of Proceedings (Bound copy).
	"Shipbuilding from its beginning," vols. 1, 2 and 3.
Permanent International Association of Road Congresses	Bulletins (Three).
	Reports of Executive Committee (Two).
	Report of Proceedings of 3rd Congress, held in London, June, 1913.
	Report on Construction of Macadamized roads bound with tarry, bituminous or asphaltic materials (Two copies).
	Report on Terminology adopted or to be adopted in each country relating to Road construction and maintenance.
Boston Transit Commission	Nineteenth Annual Report.
Master Builders' Association ...	Handbook and Diary (1914).
	,, ,, (1915).
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Surveyors' Institution	"Transactions," vol. xlvi, part 1.
Royal Society of Arts	Year Book.
Editors	The "Central" Journal, vol. x, 1913.
Institute of Metals	"Journal," vols. x, xi (1), and xii (2). Proceedings.
Liverpool Engineering Society ...	"Transactions," vol. xxxiv.
R. Feret	Pamphlet entitled "L'Imperméabilisation des mortiers et les huiles lourdes."
Constable & Co.	"Reinforced Concrete Railway Structures." By J. D. W. Hall Assoc.M.Inst.C.E. Edited by G. Moncur.
	"The Practical design of Steel-framed Sheds." By A. S. Spencer.
American Society of Mechanical Engineers	Year Book (1914).
	,, (1915).
Professional Memoirs, Washington Barracks, U.S.A.	Pamphlet entitled "A War with Mexico, 1846–48." By H. E. Haferkorn.
Leslie H. Allen	Pamphlet entitled "Aberthaw Tests on Concrete in Sea-water."
	Pamphlet entitled "Cost accounting on Concrete Work."
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Professor W. E. Lilly	"The Design of Plate Girders and Columns."
	Pamphlet entitled "The Strength of Columns."
Minister of the Interior, Quebec ...	Report of the Royal Commission on the Quebec Bridge Enquiry, 1908. Vols. 1 and 2; also Plans.
Institution of Mechanical Engineers	Proceedings, 1913 and 1914. Parts 1 and 2 (1914).
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Ewart S. Andrews, B.Sc.	Own "Alignment Charts."
Chapman & Hall	"A Treatise on Hand-lettering for Engineers," &c. By W. J. Lineham.
	"The Strength of materials." By E. S. Andrews, B.Sc.
Longmans, Green & Co.	Rivington's "Notes on Building Construction," parts 1 and 2. Edited by W. Noble Twelvetrees.

DECEASED MEMBERS

We have to announce with regret the death of the following members to the end of 1915, and not previously recorded :—

Mr. W. G. Kirkaldy, Assoc.M.Inst.C.E., Member of Council of the Institute.

Mr. F. Dare Clapham, F.R.I.B.A., unfortunately killed in a street accident in London.

M. Armand Considère (Paris), Correspondent de l'Académie des Sciences ; Inspecteur Générale des Ponts et Chaussées (en retraite).

Major A. H. Tyler, R.E., F.R.G.S. (Salisbury), killed in action, fighting for his King and Country.

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

FIFTY-SEVENTH ORDINARY GENERAL MEETING

THURSDAY, FEBRUARY 18, 1915

THE FIFTY-SEVENTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, February 18, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

MEMBERSHIP.

ASA WHITE KENNEY BILLINGS, Fellow American Inst.E.E., Member American Society C.E., Member American Society Mech.E., Member Int. Assn. Testing Materials, Vice-President and Managing Director Ebro Irrigation and Power Co., Ltd., Barcelona, Spain.

TRANSFER TO ASSOCIATE-MEMBERSHIP.

GERALD JAMES MURPHY, Stud.Inst.C.E., Asst. Engineer in Engineer Dept., Midland Railway, N.C.C., Belfast.

STUDENTSHIP.

ALBERT WILLIAM ROGERS, Junior Draughtsman, c/o British Reinforced Concrete Engineering Co., Ltd., 82 Victoria Street, S.W.

MR. T. A. WATSON, Assoc.M.Inst.C.E., M.C.I., then read his paper as follows :—

ECONOMY IN REINFORCED CONCRETE CONSTRUCTION

WHEN our energetic Secretary suggested, I should read a paper before the Concrete Institute, it was with a certain amount of misgiving that I agreed to his suggestion, as I had not a vast amount of spare time in which to write anything, and I was inclined to think that many, if not most, of you know more about the subject of Economy in Reinforced Concrete than I do. Still there may be some, to whom some of the information is fresh, and the paper may help by discussion to bring forth from the members of the Institute points of economy which are unknown to all of us, and so I trust, that it will prove of some benefit to the Institute.

You will notice that the paper is supposed to be on Economy in Reinforced Concrete, which would leave the broader aspect of the Economy of Reinforced Concrete untouched, and would confine the discussion within limits which I did not intend ; so with your permission I shall touch on points in the larger sphere, though I may be attempting to make you acquainted with things which you already know.

As you are aware, the London County Council have framed a set of regulations dealing with the construction of buildings entirely in reinforced concrete, specially with a view to the construction of buildings with their external walls of the material, it being supposed that a great economy in building construction would thereby result. I regret to say, however, I have not found that this result has been achieved.

After the new General Post Office had been constructed and Sir Henry Tanner had read a paper on it at the Royal Institute of British Architects, the view was expressed by many architects that a new era had arrived in construction ; and so it had, in a way, but not, I think, in the way that most of them seemed to believe.

It was quite true that by adopting reinforced concrete for the General Post Office Sir Henry Tanner had saved a vast amount of public money, some £60,000 in actual construction cost and some £50,000 additional for the value of extra floor space obtained by thin walls, and it was an achievement of which any one might well be proud ; but perhaps the chief value of this building to posterity was the way in which it showed that reinforced concrete could be used more economically still.

I have been going through some figures obtained during the construction of this building, and I find that the cost of some of the cheaper external walls amounted to 11s. 6d. per yard super. This cost was obtained by taking the amount of steel in a wall panel between columns and between beams in two successive floors the amount of concrete and centering, and it exceeded the cost of a 9-in. brick wall by about 2s. per yard super, though it was probably 20s. per yard super less than the cost of a wall constructed in accordance with the London Building Act in force at that time.

Of course prices are different now, but taking stock bricks at 34s. per 1,000 and picked stock facings at 39s. per 1,000 the cost of a 9-in. wall may be made up as follows :—

	s.	d.
96 bricks in 9-in. wall per yard super at 34s. per 1,000 ...	3	3
Extra over 48 bricks for facing at 5s. per 1,000	0	3
$\frac{1}{16}$th yard cement mortar, 1 : 3	1	3
Labour laying	1	9
" pointing one side only	1	3
Total per yard super	7	9

For a yard super of reinforced concrete panel wall 4 in. thick not rendered outside—

	s.	d.
3 cub. ft. of concrete, including labour of mixing and placing and part placing steel at 1s. 2d.	3	6
13 lbs. steel per yard super at 1½d.	1	8
Centering both sides, one side wrought other side left from saw	4	9
Total per yard super	9	11

You will therefore see that 9-in. brick walls are cheaper than 4-in. reinforced concrete walls.

Doubtless you will think this has nothing to do with economy in reinforced concrete construction. Well, perhaps

not, but what I wish to point out is that, though we are shortly to have regulations allowing us to build external walls in reinforced concrete, things have progressed slightly in recent years, and a reinforced concrete framed building with 9-in. or even 14-in. external panel brick walls is even more economical than the original conception of reinforced concrete external walls.

In a way this is disappointing, because it destroys an outlet for the genius of the architect to incorporate in his design the expression of the method of construction in reinforced concrete, leaving him only a skeleton to clothe in brick, stone, marble, tile, or other building material, but to my way of thinking it is true economy.

There is, however, another aspect of walls which I should like to touch on, and that is retaining walls, and particularly retaining walls outside a building. Till quite recently these were built either as gravity retaining walls of sufficient weight and stability in themselves to sustain the earth pressure and superimposed load from pavement and roadway, in which case they were enormously thick at the base, one 20 ft. high requiring to be about 7 ft. thick, or they were built as a series of vertical arches supported by walls at right angles to them and forming a series of dark dismal vaults.

In both cases the outside of these walls next the earth was lined with asphalt or a vertical damp course of some kind. At the present time reinforced concrete walls are being constructed in many cases without a vertical damp course, which even in damp ground is unnecessary. In passing let me say, however, that in waterlogged strata I personally am of opinion that a vertical damp course is absolutely necessary even with reinforced concrete walls.

But to compare prices and to give gravity walls the best possible chance let us put the average thickness of a wall 20 ft. high at 5 ft. 3 in. thick. In a wall of this thickness there are per yard super—

	s.	d.
672 bricks at 34s. per 1,000	22	10
Extra over 48 bricks for facing at 5s. per 1,000	0	3
$\frac{7}{16}$th yard cement mortar, 1 : 3	8	9
Labour laying	10	6
Pointing one side only...	1	3
Total	43	7

Leaving out the vertical damp course a reinforced concrete wall, including all counterforts and without any horizontal

struts to the main building, can be built for 32s. per yard super. The price is made up as follows :—

		Per foot-run of wall. s.	d.
Cub. ft. of concrete : 27·3, at 1s. 1d.	...	29	7
Lbs. of steel 105 : at 1½d. ...	...	13	1
Centering : 110 ft. super, at 3d....	...	27	6
		70	2 for 2$\frac{2}{9}$ yds. super

or 32s. per yard super ;

showing a saving of at least 12s. per yard super ; and a reinforced concrete wall with counterforts and struts to main building can be built for 24s. per yard super, or nearly half the cost, leaving out entirely the cost of a vertical damp course which is a necessity on all brick walls, but which is not always necessary on a reinforced concrete wall.

In addition to the above saving there is the extra space gained, roughly 4 ft. in width, which on a 1 ft.-run of wall would give 4 sq. ft., which at a rental value in the City of, say, 5s. per foot per annum amounts to 20s. per annum, or a capital value of £30 per foot-run of wall, which works out at a saving of

$£\frac{30}{20} \times 9 = £13$ 10s. per yard super of wall.

Of course, walls of a less height do not show anything like this saving, but in these days a basement of 20 ft. deep is not uncommon and the savings represented above can be made frequently.

After the enormous saving shown above the saving on a reservoir wall seems a mere trifle, but still it is worth having, and it is enhanced by the fact that it is unnecessary to have an asphalt lining on the inside of a reinforced concrete reservoir, as it is not impossible to make a reservoir watertight enough for all practical purposes without this lining. So that even if there were no saving in the cost of reinforced concrete walls against brick for a reservoir, yet there is a saving of 5s. to 6s. per yard super on the lining.

From the foregoing you will see that while great savings can be made on retaining walls, it is generally not economical to employ reinforced concrete for the external walls of buildings, as brick walls up to a thickness of 14 in. are cheaper, but that when walls have to be thicker than this reinforced concrete comes in. As under the proposed London County Council Regulations nearly all external walls need not exceed 14 in. in a framed building, it is probable that

reinforced concrete walls for this purpose will soon cease to be used from the point of view of economy.

With regard to the economy of reinforced concrete-framed buildings outside the London area the economy is very considerable ; inside the London area the London County Council, by their regulations, propose to reduce it as much as possible. There are, however, opportunities to effect savings over a steel-framed building even in London.

Economy of Pillars.

Take a pillar carrying, say, 200 tons central load, height 16 ft. from floor to floor, ends fixed. A steel stanchion 40 sq. in. in section, weighing 136 lbs. per foot-run exclusive of cap base and connections of girders, is required, size 14 in. by 16 in. According to the 1909 Act relating to steel-framed structures this has to be surrounded by 2 in. of concrete (making a total of 18 in. by 20 in.), which latter, to prevent its cracking, has to be reinforced with rabbit wire-netting, or spiral binding of some kind, the total cost of which figures out something like the following :—

	£	s.	d.
Steel stanchion, 16 ft., at 136 lbs. ; 2,176 at £13 per ton erected	13	0	0
Concrete surrounding same, 23 cub. ft. at 8d.	0	15	4
Rabbit wire, 80 ft. super at ½d.	0	3	4
Centering, 144 ft. super at 3d.	1	16	0
	£15	14	8

In order to make a saving on this price, and to conform with the regulations proposed, it is necessary to have a column slightly larger in section than the steel one. Suppose, for convenience, we say 24 in. by 24 in.

Allowing a stress of 600 lbs. per square inch in concrete and 16,000 lbs. per square inch in steel, the amount of steel required in a column this size is 83 lbs. per foot-run including links, binding, etc.

The price works out as follows :—

	s.	d.
4 cub. ft. concrete at 1s. 1½d.	4	6
83 lbs. steel at 1½d.	10	4
9 ft. centering at 4d.	3	0
	17	10 per foot-run

17s. 10d. × 16 = £14 5s. 4d.

This does not seem very hopeful, but it is something to start with. If we next consider beams, say one of 25 ft. span of a series at 20 ft. centres, load $2\frac{1}{2}$ cwt. per sq. ft. of floor, including weight of floor. A steel girder consisting of, say, one 14 in. by 6 in. and four 10 in. by $\frac{1}{2}$ in. plates, top and bottom, weight 196 lbs. per foot-run, is required. This, when covered with the requisite amount of concrete, gives a depth of beam below floor level of 22 in. and a width of 14 in.; and for the sake of making as near a comparison as possible, take a reinforced concrete beam of the same depth from floor to soffit of beam. A beam of this depth allowing 600 lbs. proposed stress in concrete and 16,000 lbs. stress in steel requires 110 lbs. of steel reinforcement per foot-run, making allowance for lappage of bars and shear members, and as regards weight of steel this compares very favourably with the steel girders ; but more of this later on.

To fill in the floor space 25 ft. by 20 ft., most architects nowadays would probably use some kind of hollow tile reinforced concrete floor, and I am of opinion they would be right ; but on the other hand there are some who would use R.S.J. fillers, and the most economical way would then be to divide the 25 ft. up into 3 bays with two R.S.J. 15 in. by 6 in. at 8 ft. 4 in. centres and use 5 in. by 3 in. by 11 lb. R.S.J. fillers at 3 ft. centres from 15 in. by 6 in. joist to 15 in. by 6 in. R.S.J., and a thickness of floor of 7 in. Comparable with this would be a reinforced concrete slab $5\frac{1}{2}$ in. thick supported by secondary beams at same centres as the 15 in. by 6 in. R.S.J. and supported by the main beams as above, Reinforcement required in $5\frac{1}{2}$ in. slab amounts to 15 lbs. per yard super. The size of secondary beams would be the same as the 15 in. by 6 in. R.S.J. in above case, viz. 10 in. in width and 20 in. in overall depth, and the steel reinforcement required in latter would amount to 24 lbs. per foot-run including lappage, etc.

Now, working out the cost in the two cases in a piece of floor 25 ft. by 20 ft., we have for the steel-framed structure :—

	£	s.	d.
One girder 25 ft. long at 196 lbs. per foot-run, $43\frac{3}{4}$ cwt., at £12 10s. per ton erected	27	6	10
Concrete casing to girder, 34 cub. ft. at 8d.	1	2	8
Shuttering to ditto, 92 ft. super at 3d.	1	3	0
Rabbit wire, 62·5 ft. super at $\frac{1}{2}$d....	0	2	7
	£29	15	1

For the reinforced concrete-framed structure :—

	£	s.	d.
One girder 25 ft. long, 110 lbs. at 1½d.	17	3	9
Concrete below floor slab, 40 ft. cube at 1s. 1½d. ...	2	5	0
Formwork, 100 ft. super at 4½d.	1	17	6
	£21	6	3

For the steel-framed structure :—

	£	s.	d.
Three girders 20 ft. long at 59 lbs. per ft., 31½ cwt., at £10	15	15	0
Concrete casing to girder, 63 cub. ft. at 8d.	2	2	0
Shuttering to ditto, 180 ft. super at 3d.	2	5	0
Rabbit wire, 90 ft. super at ½d.	0	3	9
	£20	5	9

For the reinforced concrete structure :—

Per foot-run of beam.	s.	d.
24 lbs. steel at 1½d....	3	0
1·04 cub. ft. at 1s. ...	1	1
Forms, 3½ ft. at 4½d.	1	3
	5	4 = £16 for 3 girders 20 ft. long

For the floor slab in steel-framed structure, 25 ft. by 20 ft. by 7 in. thick :—

	£	s.	d.
5 in. by 3 in. at 11 lbs., 17½ cwt. at 9s. 6d. cwt. ...	8	3	11
3 in. by 3 in. L at 7 lbs., 7½ cwt. at 10s. cwt.	3	15	0
292 cub. ft. concrete at 7d.	8	10	4
500 ft. super shuttering at 2½d.	5	4	2
	£25	13	5

For the reinforced concrete floor slab, 56 yd. super :—

Quantities per yard super.	s.	d.
4·1 cub. ft. concrete at 10d.	3	5
16 lbs. at 1½d.	2	0
Formwork	2	6
	7	11

or 56 × say 8s. = £22 8 0

Summarizing we get—

Steel-framed structure :—

£	s.	d.
29	15	1
20	5	9
25	13	5
£75	14	3

Reinforced concrete structure :—

£	s.	d.
21	6	3
16	0	0
22	8	0
£59	14	3

Saving, say, £16 0s. 0d. on a piece of floor 25 ft. × 20 ft.

From these figures you will see that it is possible, allowing for variations in design, to make a saving of 10 to 20 per cent. by constructing in reinforced concrete instead of in steel.

I am rather afraid many among you will think this is all rather elementary. I've no doubt many here could have given much better examples, and will be able to find fault with the one I have given, but my chief reason for giving you these figures is because at the present day there are many buildings being built in London as steel-framed structures which could more economically be built in reinforced concrete, and that therefore there must still be many people in London who have yet to be convinced of the desirability of the latter form of construction. I trust therefore you will excuse the detail in which I have gone into this. Other cases may of course not show quite so favourably, but on the other hand there are cases which may show more favourably.

There is another factor which enters into the question of building operations of all kinds, and that is time, and I mention it in this connection because many people are of opinion that the quickest form of construction is steel construction ; and if drawings have been prepared a long time in advance so that the steel framework has been fabricated beforehand and is ready for delivery the moment it is required it may be that the steel frame is the quicker, but this is not always so ; and starting from the same point, that is the time when working drawings of both reinforced concrete frame and steel frame are ready, generally some time after the foundations of the building have been started, it is possible in ordinary times to obtain steel bars in 14 days from that date, but it generally takes 6 weeks to obtain built-up stanchions, so that at the beginning reinforced concrete gets a 4 weeks' start.

Again, in steel-framed buildings many architects use coke breeze concrete or broken brick concrete between the

R.S.J. fillers. Neither of these are watertight; reinforced concrete 4 : 2 : 1 is practically so, and in a building constructed some years ago 7 stories high, I remember the plaster work was started on the ceilings of floors below the 4th before the 7th floor was finished, and as each of the remaining floors took about a fortnight to do, another month or six weeks was gained on the steel-framed building as asphalt on the roof would have been necessary to prevent rain coming through the breeze or broken brick floors and spoiling the plaster.

While there are still many to be converted to a belief in reinforced concrete there are some enthusiastic admirers of the material who desire to use it in every possible way, and on every possible occasion, and one of the ways in which it is used in which I am not convinced of its economy is in pitched roofs which need formwork both sides for the concrete, and asphalt or some other waterproof covering to keep the rain out. These roofs have generally very thin slabs, say 3 in. thick at most, and then the concrete costs a fabulous amount to get it in place, especially if the building has a series of saw-tooth roofs running from side to side or end to end with no flat passage-way to take concrete to the centre. These 3-in. slabs are generally supported by very light reinforced concrete trusses with sometimes diagonal members, and the roofs altogether constitute a nightmare to the contractor, who in a thoughtless moment undertook to construct one. Roofs like these should last for ever, so there may be economy in the long run, though thev ought to cost more in the first instance than steel trusses, etc.

It is possible there may be some enthusiast here, if so it would be interesting to hear his views.

It seems hardly necessary to mention that in the construction of bridges, say up to 300 ft. span at least, reinforced concrete is nearly always the most suitable material. There are of course exceptions, but the mere question of the cost of maintenance of a steel bridge seems to me enough to condemn it; and this, coupled with the fact that the life of many wrought-iron bridges constructed in this country has been found to be not longer than 65 years, and steel could not be expected to last any longer, is a great argument in favour of the reinforced concrete bridge which requires no maintenance and becomes stronger as it grows older.

Apart, however, from the question of maintenance, the saving effected in cost of construction of bridges in reinforced concrete over steel bridges is sufficient to justify its adoption,

as much as 40 per cent. having been obtained in a bridge of 42 ft. span by 26 ft. wide between parapets.

The question of maintenance is also enough to decide an architect or engineer to choose reinforced concrete for the construction of, say, small water towers, coal bunkers, gasometer tanks, or any similar structure heretofore built in steel, and exposed to atmospheric conditions, even if reinforced concrete is not cheaper in the first instance.

With regard to timber structures the same applies, and in addition the relatively greater resistance to destruction by fire makes the advantages of reinforced concrete so apparent that one wonders why timber is employed in the construction of wharfs at all.

But to turn to the other side, viz. economy in reinforced concrete, I would like to emphasize a point which in these strenuous days of competition seems to be overlooked, or rather ignored, and that is the enormous amount of waste which occurs through one of the methods adopted by architects to obtain cheap prices for the carrying out of work. An architect prepares the general design of a building and sends sunprints of same to four, five, or even six different firms of specialist designers who in turn prepare each their reinforced concrete scheme and send out to five or six or more contractors for prices, involving a total of some 30 to 40 persons, all of whom spend money and time in tendering, etc. Much valuable time is also wasted which might otherwise be spent in the construction of the building whilst the various schemes and prices are being compared and adjudicated on. This has been going on for years, and is still going on, architects and engineers going to even greater lengths and asking for all kinds of details of reinforcement, etc. This puts an initial charge on reinforced concrete which might be avoided if the architect or engineer would only choose his system, or employ a consulting engineer accustomed to the work to design the reinforced concrete work, and then send out for prices in the usual way.

Most of you know of this evil, and have known of it for years, and attempts have been made to remedy it, yet it still goes on. Is it not possible for the members of the Concrete Institute to find a remedy and so introduce another economy to the credit of reinforced concrete?

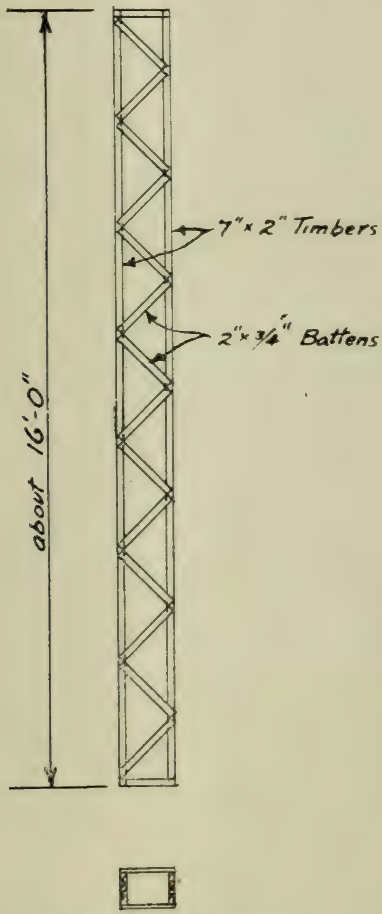
During the few years with which I have been connected with reinforced concrete many small problems have been presented to some of which solutions have been found, whilst others appear almost insoluble. One of these latter is the

question of leaving the face of reinforced concrete surfaces exposed to view free from board marks. If wrought forms thicknessed are used you have in the case of a wall still to put up one side of the wall board by board, and as you fill up with plastic concrete, the pressure at the bottom of a 3-ft. lift is greater than at the top, and slight lateral deflection is sure to occur on these separate boards, and marks will appear. There is no economical remedy for this that I know of ; more massive forms may be used, but to completely obliterate marks they would require to be of such a massiveness that expense would forbid their use, and I would suggest that architects should endeavour to persuade themselves that it is right for board marks to be there as they are an expression of the method of construction, the same as the mortar is in brick joints, and with a little perseverance in the practice of persuading themselves I have no doubt they would eventually decide that the appearance of board marks had as pleasing an effect as bricks.

There is another difficulty with centering, and that is the amount of props required, and if this is left to the foreman in charge of the work it is very likely that the number put in is either too many or too few, and I would suggest to contractors, what has often been suggested before, that great economies might be effected by employing a competent man to design the formwork. For a steel viaduct supported on trestles of any given height there is a spacing apart of the trestles which requires the use of the least amount of material, and so with props and centering, and this minimum amount of material varies according to the height of the floor and the thickness of the floor, and must be determined for each special case. I cannot give you any golden rule for obtaining the correct spacing of props to give you this minimum amount of material, as each case must be arranged for on its merits.

I have often heard it said that when floors repeat time after time in a building centering can be used over and over again and great economy result, whilst if a slight variation is made in the width of the beams and size of columns, extra expense is incurred. Now this latter statement is not always true ; props under main and secondary beams must be left whilst succeeding floors are constructed, so in any case fresh beam bottoms have to be used on the higher floors, and no extra beam bottoms are required. In order to facilitate the striking of beam sides it will be advantageous when constructing these to cut them shorter than the distance between

the centering to the sides of other beams or columns against which they abut. Fill in the space so left with a loosely fitting strip battened to the side of beam, and put in the



Sketch of Compound Strut.

usual angle fillet at the junction of beams. When striking the side of the beam take off the last-mentioned battens, remove the loosely fitting strip, and the side of the beam can be easily removed intact. If some similar method be not

adopted and the beam centering is made a good fit, the water in concrete makes the wood swell and jam, with the result that many beam sides are broken or pulled to pieces in being taken down, necessitating the expense of remaking and possibly new material. If the main beams on a higher floor are therefore 1 in. or 2 in. narrower than on the floor below it only means that a slightly wider filling-in piece at the end is required.

For stories of heights up to say 12 ft., scaffold poles are probably the most economical form of prop, but there is a limit to their size, and it frequently occurs when stories exceed 12 ft. that scaffold poles are not useful, and when this is so, instead of using die square timber, a cheap and economical strut can be obtained by using two 7 in. by 2 in. or 8 in. by 2 in. spaced, say, 6 in. apart and forming them into a lattice-braced compound strut with 2 in. by $\frac{3}{4}$ in. slating battens nailed to their edges (see illustration, p. 12).

The 7 in. by 2 in. are scarcely damaged, and the props are light and easily handled, and for their weight much stronger than a solid timber of equal section.

Another point which makes for economy in reinforced concrete, but has more to do with design, is the number of steel bars that have to be handled. It will be readily understood that it is just as easy to handle a $1\frac{3}{8}$ in. diameter bar 25 ft. long as it is to handle one that is only 1 in. diameter, as the weight is immaterial ; each requires two men to fix it in a beam, and eight bars take twice as long to put in as four, so that from the point of view of economy the four $1\frac{3}{8}$ in. which gives the same tensile resistance as the eight 1 in. should be used in preference.

Again in this connection I would point out that the $1\frac{3}{8}$ in. diameter bars occupy less space in the width of a beam than the 1 in. diameter bars, and in a case where they are used as compression bars and would be in the top of the beam, the fact that they occupy this less space makes it easier for the concrete to be placed in the beam. In fact, this labour cost on concrete may be reduced by 30 to 50 per cent. by the employment of the larger diameter bar.

Again with the four instead of eight bars there are half the number of bends, half the number of hooked or fish-tailed ends, and in many cases half the number of stirrups or shear members to fix, and the smith's work is correspondingly reduced.

I have only covered some of the ground there is in this question, but I trust that what I have said will be of some

use, and if it will promote an interchange of ideas, as I trust it will, some interesting information may be forthcoming in the discussion.

DISCUSSION

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., etc.) :—Although this is a short paper, the author has put before us some interesting figures which should be carefully considered. What we have to do is to impress the public with the fact of the economy *of* reinforced concrete and to impress ourselves with economy *in* reinforced concrete, and then the former economy will be improved by the latter. Compared with steel-frame buildings, of course there is generally a considerable saving in time, because the reinforcing metal is invariably kept in stock, while for the steel work the larger beams and stanchions have to be made to order, and very often the designer has not considered which sections can be obtained from stock and which will have to be waited for until the rolls are next put in. A considerable saving of time may be effected by observing that and by utilizing stock sections. I am glad the author again calls attention to the waste of energy and money, caused by the architect or building owner inviting competitive schemes from a large number of specialists, instead of taking counsel from one only, or, at any rate, from a very limited number. Looking through the paper, I noticed one error which I think the author overlooked in reading it. It is on page 12, in the fourth line from the bottom ; the word "no" should be deleted. Should it not read : "Fresh beam bottoms have to be used on the higher floors, and therefore extra beam bottoms are required"?

MR. WATSON :—You may read it ambiguously, Mr. President.

THE PRESIDENT :—The fact that fresh beam bottoms have to be used on the higher floors would show that extra beam bottoms are required.

MR. WATSON :—But I meant that no extra beam bottoms would be required in one case over the other.

Mr. PERCY J. WALDRAM, F.S.I., opened the discussion, and Mr. F. L. BROWN, M.C.I., followed.

Mr. EWART S. ANDREWS, B.Sc., M.C.I. :—It seems to me that the paper assumes good design on the part of the reinforced concrete designer, and rather points out failings of steelwork, which a good steelwork designer would not make. For instance, the statement of the President as to the desirability of choosing the sections, which can be readily obtained seems to me to be rather a point which, of course, must be assumed to be taken into account by the steel designer, and it would not be a point to advocate in favour of reinforced concrete as opposed to the steel design. The only thing I would like to say with regard to the paper as a whole, by way of criticism, is the reference on page 10 to steel bridges. That is the statement, that wrought-iron bridges, constructed in this country have been found to last not longer than sixty-five years. That suggests, that steel and wrought-iron undergo a process of deterioration, and that I believe has not really been proved. It seems to me, that the reason why wrought-iron bridges have to be replaced is, that the traffic has increased so much, that the design is no longer really strong enough ; or perhaps, on the other hand, that sixty-five years ago the amount of professional knowledge on the subject was such, that the design might not have been carried out quite so accurately. But I do not think that we ought to suggest, that steel, so long as the stress does not exceed the elastic limit, goes through a process of deterioration.

Mr. MORGAN E. YEATMAN, M.A., M.Inst.C.E., M.Am.Soc.C.E. (Member of Council C.I.) :—In advocating a smaller number of large rods in preference to a larger number of small rods over the same area, there is one precaution which should be taken, and that is not to use too large rods in a short beam, because you must have sufficient area in proportion to the section of the rods for the strength to be taken up from the surrounding concrete. As to the life of wrought-iron bridges, sixty-five years is about the present age of some of our greatest railway bridges,

and it is rather remarkable, that they are still in use, not so much on account of the decay of the material as on account of the increase of the rolling loads; and it is probably only where the dead load bears a large proportion to the live load, that bridges can have remained so long in service. In the United States the increase of rolling loads has been still more rapid, and some bridges have been replaced *twice* within the last thirty years for this reason.

MR. ARCHIBALD SCOTT, A.R.I.B.A., M.C.I., and MR. HABIB BASTA, Assoc.M.Inst.C.E., M.C.I., also spoke.

MR. W. A. GREEN, M.A., B.Sc.Eng., A.M.I.C.E., M.C.I. :—In listening to the author and to Mr. Waldram I think the point, that most struck me was the power of figures. I believe, that if either of them had set out to prove just the contrary he would have had no difficulty in doing so. I have no really helpful remarks to make; but I come here searching for information, and I should like to ask a question about the stresses in the reinforced concrete column given on page 6—viz. 600 lbs. per square inch in the concrete and 16,000 lbs. per square inch in the steel. I suppose that was not a London column, was it? I don't think 600 lbs. would be allowed in London.

MR. WATSON :—It was suggested in London.

MR. GREEN :—I don't pretend to know much about reinforced concrete, but in designing don't you limit the stresses in the steel to 15 times the stresses in the concrete? Well, 15 times 600 is not 16,000.

MR. WATSON :—You are quite right. The 16,000 should not be there. The 83 lbs. of steel in the column there is really stressed at times 9,000 lbs., or 15 times 600.

MR. GREEN :—Call it 4 tons.

MR. WATSON :—Yes, that is so.

MR. GREEN :—On page 5 we had a reference to a great saving in value of 5s. per square foot per

annum. Why don't we have this saving shown in that column comparison on page 6? There is a difference of $1\frac{1}{2}$ square feet between the two columns. I was much interested in the comparison between the filler joists and the ordinary reinforced concrete slabs, and Mr. Waldram has made the point, I think, that it is not the use of floors, it is the abuse of them, with which one has to deal. When you load your floors in course of construction to so many tons per foot super (quoting Mr. Waldram), the reinforced concrete slab and the tile slab I am afraid would have a poor chance; but it is wonderful what a rolled steel joist will stand! Referring again to Mr. Waldram and the Office of Works, I was much interested to hear that in London you can use a filler joist $1\frac{3}{4}$ in. wide, because from my experience the London County Council would not have allowed it, and I should have had to spread the $1\frac{3}{4}$-in. flanges to 3 in. as a minimum; but the Office of Works has ways of its own, and in this case I am in entire agreement with it.

Mr. T. C. DAWSON, M.C.I.:—I was interested in the reference to the life of bridges. I believe that the great trouble in getting reinforced concrete introduced by railway engineers for railway purposes is because they are so nervous about the material, and it would be well, I think, if the Concrete Institute would try to disabuse them of that nervousness. It is of no use a material being economic, if you cannot persuade engineers to use it. We know very well, that in the case of metal bridges, which have had a long life the weakening of the bridge is not so much due to any deterioration of the quality of the metal as to over-stressing, due to the great increase in the loads and their frequency. Railway engineers believe, or fear, that if you had a reinforced concrete bridge over-stressed, as many metal bridges are over-stressed, they would yield very much sooner, and they cannot place the same reliance upon the more brittle reinforced concrete. They are also very apprehensive of the effects of vibration in reducing cohesion. I think these are points to which it behoves us to give our attention, to see if we cannot do something to

give confidence in reinforced concrete structures as against the purely metallic bridges. In the one case you know what will happen within wide limits. I have known cases of metal bridges where the webs have been packed up with bits of timber, and, according to all theoretical considerations, the girder ought not to stand up. In fact, I have heard of a case where a defective web was discovered through a platelayer throwing a stone at a bird, and the stone went through the solid web !

MR. E. FIANDER ETCHELLS, Assoc.M.Inst.C.E., F.Phys.Soc., A.M.I.Mech.E. (Member of Council C.I.) :—I am able to support the author's contention that brick walls and brick panels in walls are cheaper than reinforced concrete walls.

The author has stated that " though we are shortly to have regulations allowing us to build external walls in reinforced concrete, things have progressed slightly in recent years, and a reinforced concrete framed building with 9-in. or even 14-in. external panel brick walls is even more economical than the original conception of reinforced concrete external walls." It is perhaps only fair to point out, that the draft regulations referred to will allow you to build in this economical form of construction. One of the clauses states, that " when portions of the external walls between the reinforced concrete pillars and the beams are constructed of brickwork, stonework, or plain concrete, such portions of walls shall be of a thickness not less than $8\frac{1}{2}$ in. for the topmost 20 ft. of their height and not less than 13 in. for the remainder of the height below the said topmost 20 ft." Those who desire to build in the form of construction, which we are told is cheaper than pure reinforced concrete will be at perfect liberty to do so.

With regard to the remarks made by Mr. Waldram, he used a phrase, which caught my ear. He asked whether the greater economy in the General Post Office building was an economy over what would be required by the " antiquated " requirements of the London Building Act. That was the phrase, and there are several answers to that question. First, the General Post Office is exempt from the constructional require-

ments of the London Building Acts. The next answer is, that many tenders have gone to show that compliance with the older London Building Act of 1894 can still be the cheapest method for certain types, sizes, and proportion of buildings, and if it may still be the cheapest under frequently occurring circumstances, then the epithet is not deserved.

In answer to Mr. Green's question, the draft regulations require that the compressive stress on a 1 : 2 : 4 concrete should not exceed 600 lbs. per square inch, and the stress on the steel in the same pillar should be 9,000 lbs. per square inch.

Mr. J. F. WARDEN, M.C.I. :—Mr. Watson spoke of the fearful expense to designers in having to make so many designs in competition and that then they might not get the job. Has he ever heard of steel-work contractors having to make designs for competition, and full working drawings, too, and not getting the job?

Mr. WATSON'S reply :—The comparison in the paper of various costs was not between what could be done by the most skilful designer with a free hand in each case, but between what could be done under the regulations governing the construction of steel buildings in London and the proposed regulations to govern the construction of reinforced concrete buildings. If an isolated case is taken of a building, which is exempt from the Building Act, there is no doubt greater cheapness could be achieved on a steel-framed building than in the case shown in the paper, but the same could also be done with a reinforced concrete-framed building.

The regulations referred to provide in both cases for a factor of safety of 4, but whereas the factor of safety of a reinforced concrete building increases with age, owing to the increasing strength of the concrete, that of the steel-framed building decreases owing to the corrosion of the steel.

There are other ways of economizing in reinforced concrete, not mentioned in the paper. One is to leave out some of the steel bars; another is to buy as little timber for centering as possible, which generally leads

to striking the centering too soon; neither of these is to be recommended. Another is the ignorance of the contractor inexperienced in the cost of reinforced concrete, who prices his labour too low, with the result that his client gets a cheap job, while the contractor gets no profit. This economy disappears as the experience of the contractor increases.

FIFTY-EIGHTH ORDINARY GENERAL MEETING

THURSDAY, MARCH 4, 1915

THE FIFTY-EIGHTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, March 4, 1915, at 7.30 p.m.,

MR. H. D. SEARLES-WOOD, F.R.I.B.A., M.R.San.I. (Vice-President C.I.), in the Chair.

MR. R. GRAHAM KEEVILL, A.M.I.Mech.E., M.C.I., read a paper as follows :—

SOME NOTES ON WIND PRESSURE

A WRITER in the *Encyclopædia Britannica* states that wind pressure is "supposed to interest the engineer and the navigator"; if we drop the latter and substitute the architect we can say that it is supposed to interest this Institute.

Wind, as we all know, is caused by the horizontal, vertical, and rotatory movements of the atmosphere which are set up by the alterations of density, temperature, vapour, and possibly gravity. When we consider the fact that the atmosphere is composed of a mixture of the gases oxygen, nitrogen, carbonic acid gas, helium, neon, argon, ammonia, together with traces of hydrogen and hydro-carbon and a greatly varying quantity of vapour, and that its movements are obstructed by its own viscosity, the friction caused by passing over land and sea and artificial erections which set up great irregularities in the travel of the wind so that velocity and pressure vary over quite small areas, we see what a very complex subject has to be dealt with.

The atmosphere as far as present-day knowledge carries us extends upwards from the surface of the earth, gradually becoming rarefied until it merges into the eternal interstellar space. Luckily, we are only concerned with movements

of the air comparatively near the earth, otherwise our subject would be vast indeed.

It may be said that our modern knowledge extends backwards only so far as the destruction of the original Tay Bridge in 1879. This bridge was part of the scheme for bridging the two firths—Forth and Tay—which was brought to a successful completion by the construction of a new Tay Bridge and the erection of the Forth Bridge ; the latter, by the way, has been opened for traffic for 25 years to-day.

It will be interesting to recall, briefly, the disastrous Tay undertaking as far as it concerns us, and see what the engineers of those days—35 years ago—knew about wind pressure.

The bridge was opened for traffic on May 30, 1878, the total cost being about £350,000. It consisted of lattice girders, with one exception of large span ; carried on groups of cast-iron columns braced together, which in turn were carried on masonry foundations. The rail level at the highest point was 88 ft. above high-water level, and there were altogether 85 spans.

On December 28, 1879, whilst a train was on the bridge and a "gale of exceptional severity was blowing," one or more of the piers gave way, bringing down eleven 245 ft. span and 227 ft. span girders, precipitating the train and 74 people into the Tay. A Board of Trade Inquiry was held, at which the following evidence was given :—

Mr. Law calculated that 36·81 lbs. per sq. ft. would blow over the bridge with everything perfectly sound and rigid, but the piers must have been very far from rigid in consequence of the imperfect manner in which the struts and ties were connected with the columns, and that that was the immediate cause of the disaster.

Mr. J. Brunlees, M.I.C.E., stated "the wind pressure in a structure such as the Tay Bridge was not generally treated of in engineering works. It was entering upon unknown ground, and so far as he knew they were all in the dark on the matter."

General Hutchinson, Board of Trade Inspector, "had no data whatever as to wind pressure—10 lbs. distributed over a large surface was an amount which might reasonably be expected —it had not been customary for civil engineers in this country to consider that subject, especially in an open structure."

The Engineer for the Bridge "made calculations, and finding that the bridge was designed to bear twenty-five times the

expected weight, he believed that there was sufficient strength to resist the force of the wind."

The Secretary of the Meteorological Office said, "that about the time of the accident, the greatest velocity registered at Glasgow in any consecutive 60 minutes was 71 miles, with greater velocities for shorter periods, the greatest of which was estimated to be 120 miles per hour."

At Aberdeen 96 miles per hour was registered for five minutes.

Sir B. Baker thought that the bridge was amply strong for the wind that it had sustained since erected—and that the failure was due to something in addition to mere statical wind pressure. In his judgment the wind pressure on the bridge as a whole (not upon an isolated part of it) did not on the night of the accident exceed 15 lbs. per sq. ft. For practical purposes, and for areas of reasonable size, he looked upon 20 lbs. as about the maximum in this country: there was no understanding among English engineers as to the wind pressure to be provided against. In his practice for works in this country he assumed the effective wind pressure to be 28 lbs. per sq. ft.

It was stated in the Commissioners' conclusions that "engineers in France made an allowance of 55 lbs. per sq. ft. for wind pressure, and in the United States an allowance of 50 lbs."

These extracts and other evidence given show that the knowledge of wind and of wind pressure at this period was in a state of chaos.

An endeavour was at once made by a Board of Trade Committee to put matters on a satisfactory footing. After examining wind records, they recommended on May 20, 1881 :—

"That for railway bridges or viaducts a maximum wind pressure of 56 lbs. per sq. ft. should be assumed for the purpose of calculation."

This immediately came into force for railway work, and was also adopted generally for structures on public works, although such adoption was purely arbitrary, or specified in local by-laws.

Much opposition was raised to this standard of pressure, because it was considered excessive and not justified by experience of existing structures. As far as buildings are concerned, the horizontal pressure assumed for designing has gradually been reduced to the 30 lbs. per sq. ft. now often taken.

Two questions of great importance arose, and these have since been practically settled by research, viz.—

(*a*) Were the wind velocities given by the anemometers then in use, correct ?
(*b*) What was the relation of wind velocity to wind pressure ?

There are three methods for the measurement of wind :—

(i) Velocity measured by the Robinson anemometer.
(ii) Pressure measured by a pressure tube anemometer.
(iii) Pressure measured by a plate or board set face to the wind.

Considering (i), the anemometer that bears his name was invented by Dr. Robinson in 1846, and is by far the most used of the various instruments which have been produced from time to time. It consists of four arms at right angles ; at the end of each arm is a hemispherical cup. In the standard pattern the cups are 9 in. diameter, whilst the arms are 2 ft. long. The arms are mounted on a spindle, which rotates with the movement of the cups in a wind, and is geared to mechanism for counting the revolutions made. The cups are set to face all one way, so that the concave face of one cup is always presented to the wind, and consequently as the wind pressure within the cups is greater than on their convex backs, rotation is caused by quite a small wind. This instrument therefore measures direct velocity, and Dr. Robinson, after experimenting, stated that the velocity of the cups was one-third the velocity of wind for all instruments whatever their size. This is now known to be a fallacy, with the result that practically all records of velocity prior to 1902 were incorrect and generally much too high, a fact which justifies the distrust shown by the experts at the Tay Bridge inquiry respecting the records quoted then.

The discovery that the factor, 3, for the Robinson anemometer was incorrect is due to the experiments carried out by Mr. Dines, and since confirmed by other authorities.

These experiments consisted of placing anemometers on horizontal arms, which were rotated by means of a steam engine ; 14½ revolutions of this "whirler" constituted a half a mile of space, whilst great care was taken to ensure accuracy in the results. The number of turns of the whirler

and the revolutions of the anemometers, both of which were counted automatically, determined the relation between the distance travelled and the revolutions of the anemometers, and thus the value of the new factor, 2·2, was obtained.

The outcome was that the Wind Force Committee of the Royal Meteorological Society in 1902 adopted the factor 2·2 instead of 3 for the Standard Robinson anemometer. Thus when the old factor was used, and the instrument indicated, say, a velocity of 60 miles per hour, the use of the new factor reduced the velocity to 44 miles per hour. Converting these velocities into pressures by the formula $P = 0{\cdot}003\ V^2$, the 60 miles per hour velocity gives 10·8 lbs. per sq. ft., whilst the velocity of 44 miles per hour gives a pressure of 5·8 lbs. per sq. ft. A very serious alteration, which doubtless explains the fact that chimneys, buildings, and bridges, which, by all the rules of the anemometers, should have fallen down, are standing until to-day, or meeting an honourable end at the hands of the housebreakers. This rotating anemometer has one great disadvantage, in that owing to the inertia of the instrument it does not record gusts of wind, which may have a much greater velocity than the average values shown by this instrument. For instance, if the record is, say, 37 miles per hour, it shows that the cups have passed through a distance of 37 miles in the hour ; at some part of the time they would be revolving faster than at others, but the record shows the average speed.

Another type of instrument is the pressure tube anemometer invented by Mr. Dines. This instrument automatically records all the varying pressures of the wind, and consequently all gusts and lulls that occur during a gale whatever the velocity of the wind may be. The principles upon which the design of this instrument is based are that when the wind blows into the open horizontal mouth of a tube it causes an increase of pressure within it and in all air-tight arrangements connected with it. On the contrary, if the wind blows horizontally across the open mouth of the vertical tube, the column of air within the tube rises and the pressure is reduced within it, and in all air-tight arrangements connected with it.

In Messrs. Negretti and Zambra's pamphlet describing this instrument they say that it consists of two parts :—

(*a*) The vane and head, which being of very small dimensions may be mounted on a pole or mast in the most exposed position possible.

(*b*) The recorder, which may be placed at practically any distance from the vane (not necessarily in a straight line) in any convenient room or building.

The connection between the head and the recorder is by means of two lengths of flexible lead or composition tubing of $\frac{3}{8}$-in. bore.

The head consists of two tubes, one with an open end which is always facing the wind, the other being a perforated tube exposed on all sides to the wind. The action of the wind is to produce a pressure in the former, and a suction in the latter, both dependent upon the wind velocity. The pressure and suction are conveyed to the recorder by the tubes mentioned above, and act upon the interior and exterior, respectively, of a bell-shaped float immersed in liquid and contained in a closed vessel. Attached to the float is a light spindle passing through a collar in the cover of the float container, and carrying a pen recording on a clock drum in the usual way. The movements of the float are controlled by a pair of springs attached to the pen spindle and calibrated in such a way as to convert the effects of pressure and suction (which vary as the square of the wind velocity) into a practically uniform velocity scale.

The pressure and suction effects of the wind acting upon the float gives a perfectly true record of the action of the wind, the two tubes being necessary to eliminate casual alterations of pressure in the room where the recorder is placed.

The great advantage of this instrument is that it not only gives a record of all phases of wind, but a practically perfect exposure can be obtained by mounting it on a high pole.

The third type of anemometer is like the second type, a pressure recorder, and is in general use, although now practically obsolete. This type consists of a board or plate which by means of a weather vane is kept facing the wind. The back of the board is fitted with springs which keep the board up to its work. The wind acting on the face of the board drives it backwards, compressing the springs; the pressure is then measured and recorded automatically by the movement of the board or the compression of the springs. This type of instrument has been in use for many years, and, like the Robinson anemometer, the earlier forms recorded pressures which are now known to be erroneous and much too high. The cause of the high pressures recorded up to 60 lbs. per sq. ft. and over was faulty design and the

momentum of the moving parts. The result has been that high pressures have been recorded and quoted by authorities, when as a matter of fact the actual pressure was something less than half those shown.

This type of pressure boards was used at the Forth Bridge during its construction.

When Sir B. Baker, in conjunction with Sir John Fowler, undertook the responsibility for this great structure, it became necessary to ascertain as nearly as possible what would be the probable wind pressure on the bridge in the exposed position on which it is sited.

He accordingly erected, on the top of the Old Castle on Inchgarvie, three anemometers. The principal one was 20 ft. long by 15 ft. high, approximately the size of the side of a railway carriage of those days, erected vertically with its surfaces facing east and west, this direction being the most unfavourable from which the wind can strike the bridge. This board was hung in a frame and was carefully adjusted and balanced and fitted with four springs at the four corners to adjust the board to its normal position. The movement of the board was communicated to the registering apparatus by means of wires.

To ascertain to some extent the effect of small local gusts plates were fitted 18 in. diameter in holes cut out of the centre of the board, and at the top right-hand corner, and arrangements made to register the pressure on them separately.

A circular plate with an area of 1½ sq. ft. was erected to face the same direction, and 8 ft. away from the large board, and provided with means for registering the pressure.

The third was a revolving gauge also with an area of 1½ sq. ft. and fitted with a vane so that it always faced the wind.

These anemometers, although giving good results, were not intended to show very accurate results.

The records of the violent gales are shown in Table 1, and were published in *Engineering* in 1890.

These figures show that the average on the large board is 0·565 of the average on the small gauge, and that the average on the large board is 0·592 of the average on the revolving gauge. If we round these figures up we can say that the pressure on the large board is about five-eighths the amount shown by the small gauges; a figure to be remembered when considering the records from the small anemometers in general use, and applying them in the design of buildings of large exposed area.

TABLE 1.

Date.	Pressure in lbs. per sq. ft.					Direction of Wind.
	Revolving Gauge.	Small Fixed Gauge.	Large Fixed Gauge.	Centre of Large Gauge.	Top Right Hand of Large Gauge.	
October 27, 1884 ...	29	23	18	—	—	S.W.
October 28, 1884 ...	26	29	19	—	—	S.W.
March 20, 1885 ...	30	25	17	—	—	W.
December 4, 1885 ...	25	27	19	—	—	W.
March 31, 1886 ...	26	31	19	28·8	22	S.W.
February 4, 1887 ...	26	41	15	—	—	S.W.
January 5, 1888 ...	27	16	7	—	—	S.E.
November 17, 1888 ...	35	41	27	—	—	W.
November 2, 1889 ...	27	34	12	—	—	S.W.
January 19, 1890 ...	27	28	16	—	—	S.W.
January 21, 1890 ...	26	38	15	—	—	W.
January 25, 1890 ...	27	24	18	23½	22	S.W. by W.
Average	28·41	29·75	16·83			

TABLE 2.

Date.	Positions of Wind Gauges.					Direction.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	
	Pressure in lbs. per sq. ft.					
January 26, 1901 ...	25	15	65	—	—	
November 23, 1901 ...	55	50	60	—	55	
December 13, 1902 ...	31	27·5	18	—	34	
January 10, 1903 ...	25	20	60	15	27·5	
January 31, 1903 ...	29	10·5	65	—	26	
March 18, 1903 ...	25	20	31	20	20	W.S.W.
March 21, 1903 ...	20	20	54	10	22·5	W. & S.W.
March 2[illegible], 1904 ...	32	20	52	—	27	
December 29, 1904 ...	22·5	22·5	—	—	32·5	
January 21, 1905 ...	30	21	—	—	23	
February 28, 1905 ...	20	22	38	10	20	
March 15, 1905 ...	32·5	32·5	60	—	42	
January 11, 1906 ...	23·5	20	30	10	25	
January 26, 1906 ...	—	—	59	15	—	S.W.
February 8, 1906 ...	25	15	55	10	25	

TABLE 3.

Station.	Number of Gales with Mean Velocities in Statute Miles per hour between the Limits.												Total Number of Gales.
	37 to 40	40 to 43	44 to 47	48 to 51	51 to 54	55 to 58	59 to 62	62 to 65	66 to 69	70 to 73	73 to 76	77 to 80	
	Corresponding Pressures in lbs. per sq. ft.												
	4·1 to 4·8	4·8 to 5·5	5·8 to 6·6	6·9 to 7·8	7·8 to 8·7	9·1 to 10·1	10·4 to 11·5	11·5 to 12·7	13·1 to 14·3	14·7 to 16·0	16·0 to 17·3	17·8 to 19·2	
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	
15 years—1890 to 1904.													
Deerness ...	128	78	57	17	12	4	3	—	—	1	—	—	300
Aberdeen ...	16	3	5	—	—	—	—	—	—	—	—	—	24
Alnwick Castle...	64	43	28	10	6	5	1	—	—	—	—	—	157
N. Shields ...	36	12	8	3	—	1	—	—	—	—	—	—	60
Yarmouth ...	37	26	11	8	—	1	—	—	—	—	—	—	83
Fleetwood ...	98	78	59	30	14	12	5	5	3	—	1	1	306
Holyhead ...	106	55	40	15	10	5	—	1	—	—	—	—	232
Valencia... ...	66	39	12	8	2	1	1	1	—	—	—	—	130
11 years—1891 to 1901.													
Armagh	2	—	—	—	—	—	—	—	—	—	—	—	2
Falmouth Observatory... ...	16	7	1	1	—	—	—	—	—	—	—	—	25
Kew	1	—	—	—	—	—	—	—	—	—	—	—	1
4 years—1900 to 1903.													
Kingstown ...	48	30	14	6	3	3	2	—	—	—	—	—	106
	37 to 41	42 to 46	47 to 50	51 to 55	56 to 60	61 to 64	65 to 69	70 to 74	—	—	—	—	—
	4·1 to 5·0	5·3 to 6·3	6·6 to 7·5	7·8 to 9·1	9·4 to 10·8	11·2 to 12·3	2·7 to 14·3	14·7 to 16·4	—	—	—	—	—
15 years—1890 to 1904.													
Scilly	206	143	75	41	16	7	2	4	—	—	—	—	504
11 years—1891 to 1901.													
Dublin ... ... (Phœnix Park)	6	—	—	—	—	—	—	—	—	—	—	—	6

After the completion of the bridge, five revolving gauges with a plate area of $1\frac{1}{2}$ sq. ft. were fixed on the bridge, and a continuous record was obtained. This record of gales is shown in Table 2, which Mr. Adam Hunter, M.Inst.C.E., gave in a paper he read at the Junior Institution of Engineers in 1907.

These results from the Forth Bridge are interesting and valuable in that they show records of an exposed position at considerable differences in height and width. From a consideration of them with other reliable information now available, they appear to be high in value and to be influenced by gusts and possibly by some momentum of the plates of the anemometers.

The most important information available respecting wind velocities is undoubtedly that given by Dr. Shaw in his report on the Beaufort scale of wind force, which is compiled from the records of the anemometers at various meteorological stations.

This information, which is given in Table 3, shows the number of gales of wind exceeding 37 statute miles in the hour, grouped according to the maximum hourly velocities recorded by Robinson anemometers.

In order to see at a glance the results valuable to us, the velocities have been reduced to pressures by the formula $P = 0{\cdot}003\,V^2$.

To bring this table up to date, Dr. Shaw has kindly supplied me with the information in Table 4, which shows wind velocities of force 10, or upwards, of the Beaufort scale, recorded on anemometers at stations in connection with the Meteorological Office during the years 1899 to 1913.

These records may be compared with those shown in columns 6, 7, 8, 9, 10, and 11 in Table 3.

The point that is most striking after a study of these two tables is that pressures over about 12 lbs. per sq. ft. are rare. It must be borne in mind these records are averages, and that individual gusts would give higher pressures on small areas.

It will be of interest to note that the greatest velocity recorded since the Meteorological Office has been at South Kensington is 54 miles per hour, on July 28, 1911, which is equivalent to a pressure of about 8·75 lbs. per sq. ft.

Dr. Shaw, in his Reports and Memoranda, No. 9, compiled in connection with the Advisory Committee for Aeronautics, defines "the ordinary 'gustiness' of a steady wind" as "the variation of the velocity, and direction of the wind, while the average hourly velocity remains practi-

cally unchanged, or increases, or decreases gradually. The intervals of these gusts are of the order of some fraction of a minute; they are largely a question of exposure of the anemometer."

He proceeds: "This question has been treated by Dr. G. C. Simpson in his report on the Beaufort scale of wind force (M.O. publication 180, pp. 37–40), in which he shows that for the anemograms of Scilly and Holyhead the effective

TABLE 4.

Station.	Average Velocity during an Hour or more in miles per hour.		
	55 to 63.	64 to 75	Above 75.
	Corresponding Pressures in lbs. per sq. ft.		
	9·1 to 11·9.	12·3 to 17.	Above 17.
Blackpool	—	—	None except in gusts.
Edinburgh	1	—	
Deerness	5	—	
Fleetwood	8	2	
Holyhead	1	—	
Kingstown	12	1	
Liverpool	—	1	
Pendennis	19	1	
Plymouth	1	—	
Roche's Point	1	—	
Scilly	10	4	
Shields	1	—	
Southport	4	—	
Valencia	1	—	

'gustiness' of the wind of an ordinary steady character, as estimated by the range of variation of velocity, is proportional to the velocity.

"The trace of a Dines pressure tube is, in fact, not a steady line, but the line traced by a point constantly oscillating transversely, like the woof of a ribbon. Looking at the traces, one recognizes that there is a fairly well-defined lower limit of the range of oscillation of the gust, and corresponding thereto is the mean velocity in the gust; on the other hand, one sees that there are occasional excursions beyond the limit, which suggests a broader ribbon within the limit

of the extreme of velocities for the hour. Dr. Simpson finds that the range of the mean velocity in gusts, and of the extreme velocities within the hour, are practically proportional to the hourly velocity. Calling the mean hourly velocity V, we get approximately the following results :—

Mean gust velocity	= 1·2 V
„ lull „	= 0·75 V
Extreme gust „	= 1·3 V
„ lull „	= 0·65 V

" In other words, a steady wind with an hourly velocity V will really be oscillating within a range of about 25 per cent. on either side of the mean, with occasional excursions up to 33 per cent. on either side ; or, in dealing with a ' steady ' wind of 15 miles per hour, we must be prepared to face alterations of velocity between 20 miles per hour and 10 miles per hour.

" What is well established is that the ' better ' the exposure of an anemometer the less is the range of the ordinary gusts. The breadth of the ribbon of the Dines records for a 30-mile average wind for the following stations arranged in the order of freedom and exposure, comes out as follows :—

Marshside, 10 miles	Near Southport. A dead-level plane extending many miles inland. Vane of instrument 50 ft. above ground.
Scilly, 15 miles	Instrument on top of hill of gradual slope 130 ft. high.
Shoeburyness, 10 miles (E.N.E. wind) ... ... Shoeburyness, 25 miles (W. wind)	At 100 ft. exposure similar to Southport, but houses, buildings, trees, and rising ground near.
Holyhead, 15 miles	Almost as good as Scilly, but hills not far off.
Pendennis, 8 miles (S. wind) ... „ 16 „ (W. wind) ...	Anemometer is on Castle that crowns a conical hill.
Aberdeen, 30 miles	—
Alnwick, 25 miles	—
Kew, 30 miles	Vane 70 ft. above ground, but district abounds in fine timber trees, which on one side are at no great distance, and buildings are within sight.

" It is clear that inland exposures in the neighbourhood of trees, or buildings, give much wider ribbons than coast exposures. It seems reasonable, therefore, to conclude that the gustiness shown on an anemometer record is due to the effect of the ground. There is, therefore, some reason for thinking that the gustiness will be negligible when the effect

of the ground is no longer noticeable." The level at which it is thought that gusts die out, is far above the level of any building, so that we must take into account gusts when designing structures.

In Appendix III to the Weekly Weather Report, 1913, issued by the Meteorological Office, the appended table (5) of highest velocity in a gust of storm force, *i.e.* above 55 miles per hour, during each of the years 1906–13 is given.

These velocities have been converted into pressures by the formula $P = 0{\cdot}003\ V^2$.

From Table 5 it appears that during the 8 years covered by it there has been :—

1 gust	of 98 miles per hour	= 28·8 lbs. per sq. ft.	
2 gusts	of 90 " "	= 24·3 " "	
9 "	85 to 89 miles per hour	= 21·7 lbs. per sq. ft.	to 23·8 lbs. per sq. ft.
11 "	80 " 84 " "	= 19·2 " "	21·2 " "
21 "	75 " 79 " "	= 16·9 " "	18·7 " "
19 "	70 " 74 " "	= 14·7 " "	16·4 " "
26 "	65 " 69 " "	= 12·7 " "	14·3 " "
22 "	60 " 64 " "	= 10·8 " "	12·3 " "
6 "	55 " 59 " "	= 9·1 " "	10·4 " "

It must be borne in mind that all the stations are in exposed positions, and the instruments are placed to give the best exposures.

The recent greatest gusts during gales in London are :—

TABLE 6.

Date.	Station.	Velocity, miles per hour.	Pressure, lbs. per sq. ft.	Time.	Direction.
Dec. 28, 1914	Kew	60½	11	8.25 p.m.	W.S.W.
"	S. Kensington	40	4·8	8.40 p.m.	W.
Jan. 16, 1915	Kew	63	11·9	2.45 a.m.	W.
"	S. Kensington	40	4·8	2.35 a.m.	W.S.W.
Feb. 2, 1915	Kew	56	9·4	6.18 p.m.	S.W.
"	S. Kensington	27	2·2	8.20 p.m.	S.W.

The instrument at South Kensington is sheltered to some extent by surrounding buildings.

Another type of wind is known as the "line squall" which, Dr. Shaw says, sets "in as a temporary wind of great violence, accompanied by sudden changes of all the meteorological

TABLE 5.

District and Station.	1906.			1907.			1908.			1909.			1910.			1911.			1912.			1913.		
	Velocity in miles per hour.	Pressure in lbs per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.	Velocity in miles per hour.	Pressure in lbs. per sq. ft.	Date.
1. Aberdeen	—	—	—	—	—	—	79	18·7	Jan. 6	70	14·7	Nov. 12	66	13·1	Feb. 20	70	14·7	Feb. 17	75	16·9	Nov. 26	77	17·8	Feb. 8
Rosyth	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—		—	85	21·7	Nov. 26	70	14·7	Dec. 16
2. Alnwick	—	—	—	62	11·5	Nov. 12	58	10·1	Feb. 24	66	13·1	{Nov. 12 / Dec. 3}	64	12·3	Feb. 17	73	16·0	Nov. 5	71	15·1	Apr. 8	63	11·9	{Feb. 8 / Nov. 20}
South Shields ...	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	78	18·3	Nov. 5	81	19·7	Apr. 8	74	16·4	Feb. 8
3. Spurn Head ...	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	70	14·7	Mar. 19
Gorleston	—	—	—	—	—	—	64	12·3	Nov. 23	65	12·7	Dec. 3	62	11·5	Feb. 20	64	12·3	Sept. 30	61	11·2	Jan. 17	61	11·2	Mar. 19
Shoeburyness ...	60	10·8	Jan. 6	67	13·5	Dec. 14	67	13·5	Feb. 22	72	15·6	Dec. 3	70	14·7	Feb. 20	61	11·2	Dec. 10 and 20	61	11·2	Mar. 4	67	13·5	Mar. 22
4. Pyrton Hill... ...	—	—	—	—	—	—	*	*	*	66	13·1	Dec. 3	*	*	*	60	10·8	Nov. 5	58	10·1	Mar. 4	*	*	*
5. Dover	—	—	—	—	—	—	69	14·3	Sept. 1	67	13·5	Dec. 3	72	15·6	Jan. 11	71	15·1	Jan. 12	67	13·5	Mar. 4	73	16·0	Mar. 17
Warlington... ...	—	—	—	—	—	—	—	—	—	—	—	—	*	*	*	*	*	*	60	10·8	Nov. 26	61	11·2	Mar. 22
Brighton	—	—	—	—		—	—	—	—	61	11·2	Oct. 23	68	13·9	Feb. 20 and 21	64	12·3	Dec. 10	65	12·7	Dec. 26	62	11·5	Mar. 22
Kew	*	*	*	*	*	*	58	10·1	June 1	59	10·4	Dec. 3	64	12·3	Feb. 20	*	*	*	60	10·8	Mar. 4	67	13·5	Mar. 22
6. Eskdalemuir ...	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	90	24·3	Nov. 5	78	18·3	{Apr. 8 / Dec. 24}	82	20·2	Feb. 8
7. Southport	—	—	—	81	19·7	Mar. 15	81	19·7	Feb. 22	76	17·3	Dec. 3	85	21·7	Feb. 21	80	19·2	Nov. 5	66	13·1	{Apr. 8 / Dec. 24}	86	22·2	Feb. 7
Holyhead	68	13·9	Nov. 15	75	16·9	Feb. 20	81	19·7	Feb. 22	69	14·3	Dec. 3	68	13·9	Feb. 17	75	16·9	Nov. 5	75	16·9	Nov. 10	83	20·7	Feb. 7
Dwyran	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	71	15·1	Dec. 6–7	75	16·9	Nov. 26	79	18·7	Feb. 7
8. Pendennis	85	21·7	Jan. 6	74	16·4	Jan. 2	78	18·3	Aug. 31	79	18·7	Apr. 20	87	22·7	Feb. 19	77	17·8	Dec. 10–13	98	28·8	Mar. 4	75	16·9	Mar. 15
Plymouth	—	—	—	—	—	—	58	10·1	{Feb. 27 / Mar. 9}	66	13·1	Oct. 23	76	17·3	Dec. 13	64	12·3	Apr. 18	82	20·2	Dec. 20	66	13·1	May 8
10. Quilty	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	78	18·3	Dec. 5	88	23·2	Dec. 24	80	19·2	Feb. 7
Roche's Point ...	72	15·6	Feb. 10	68	13·9	Oct. 18	70	14·7	Mar. 6	73	16·0	Dec. 2	68	13·9	Oct. 31	88	23·2	Dec. 18	66	13·1	{Oct. 21 / Dec. 24}	75	16·9	Jan. 24
11. Scilly	81	19·7	Jan. 6	78	18·3	Feb. 20	84	21·2	Feb. 28	90	24·3	Oct. 23	86	22·2	Feb. 20	73	16·0	Dec. 13	88	23·2	Dec. 20	72	15·6	Dec. 4

* No gusts of storm force.

elements, and generally also by thunder and lightning. These are the most dangerous of all winds, with the exception of tropical hurricanes, which do not occur in our latitude.

"The following examples are cited :—

March 24, 1878, *Eurydice* squall.
February 8, 1906.
August 2, 1906, very destructive in Hampshire.
June 1, 1908, destructive at Bushey Park."

The record for the latter at Kew is shown in Table 5.

"These line squalls traverse the country with a line front like the front of a tidal wave, with a speed approximating to that of an express train, and when once identified their onset at any particular locality can sometimes be predicted with great accuracy. In less marked form they are of comparatively frequent occurrence."

The relation between coast and inland winds is a practical question which often arises when an important structure is to be designed in some locality where building regulations are of an elementary nature, or non-existent, and where the only guide is the possession of reliable information, such as that already quoted, but which has been taken at one or more exposed stations near the coast, whilst the proposed structure may be sited some distance inland. Dr. Shaw says that, "speaking from the daily experience of weather maps we may say that a westerly wind of force 8 at an exposed western station will be represented at inland or east coast stations by a wind force of 5. The corresponding reduction of velocity is from 42 to 21 miles per hour. Provisionally, therefore, we may say that the surface wind velocity" (that is winds that will affect a structure) "at a reasonably well-exposed inland station, or at a coast station for an off-shore wind, is reduced to one-half the velocity of an open coast station." As pressure is dependent on velocity it follows that the pressure will be reduced to a corresponding amount.

When dealing with a building of large exposed area, there is often a difference of opinion, in the absence of regulations, as to what wind pressure to adopt, *i.e.* whether the maximum average velocity, or the greatest gust. There is little available information as to what may be termed the area of a gust, but it is considered to be comparatively small. In a well-designed and well-constructed structure, it would appear to be good enough to take the maximum velocity, and trust

to the continuity of the structure to distribute local gusts. In Professor Henry Adams's engineers' handbook there is a table for reduction of wind pressures according to heights and widths of a building.

The knowledge of wind at low, and high altitudes—the latter concerns the flying men more than anybody else—has made great progress in recent years. Investigations on a large scale have been carried out by Dr. Shaw's department, and it is now possible to calculate the velocity of the wind at high altitudes, when the surface velocity is known. "As a rule (but with exceptions) the wind increases rapidly as the surface is left, up to two, three, or four times the surface velocity. The increase of wind appears to be in direct ratio with increase of height.

"For the station at Ditcham Park it has been found that—

$$V_h = \frac{h}{h_0} V_s$$

where V_h = velocity at height h above sea-level,
h_0 = height of a well-exposed anemometer above sea-level,
V_s = velocity recorded by the anemometer.

"The corresponding law for other places is not yet known. There is, however, a consensus of opinion that velocity increases with height, by factor and not by constant addition. A likely formula is :—

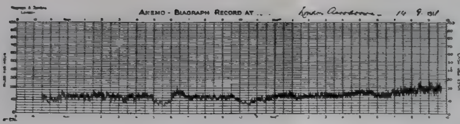
$$V = \frac{H + a}{a} V_s$$

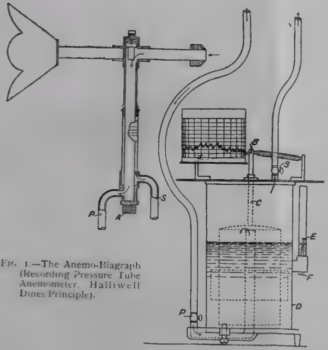
where H = a height above ground,
V = velocity at H,
V_s = velocity recorded by the anemometer,
a = constant.

"From the examination of observations at various stations, the following curious conclusion has been drawn : The constant a for the purposes of rough approximation may be taken to be *the height of the station above sea-level.*"

Another method of obtaining a knowledge of the wind is by a system of observation.

In 1805 Admiral Beaufort, Hydrographer of the Navy, devised the scale of wind force which bears his name. The scale originally had no direct relation to the velocity of the





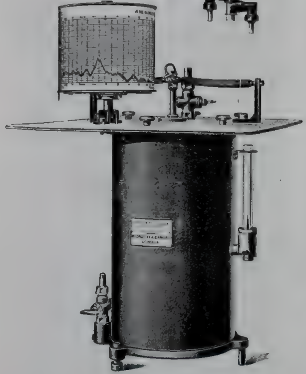
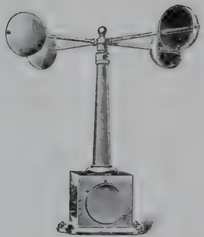
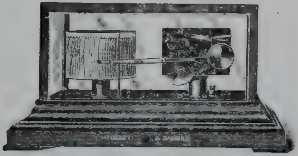


Fig. 1.—The Anemo-Biagraph (Recording Pressure Tube Anemometer. Halliwell Dines Principle).

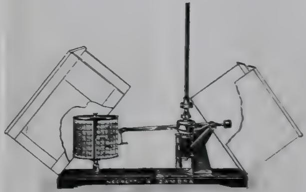


Transmitter.
(Height, 17 in. ; width over cups, 17 in.)



Recorder.
Fig. 2.—Lowne's Patent Electrical Recording Anemometer.

Length, 4 ft 6 in.



Size, 21 in. by 9 in. by 7 in.
Fig. 3.—Halliwell's Patent Wind Direction Recorder.

wind, its lower numbers being based on the speed of a frigate (man-of-war), and the numbers indicated what amount of sail could be carried ; whilst number 12 indicated a hurricane at which no sail was possible but was not necessarily the greatest possible wind.

With the passing of the sailing ship and the present universal use of steam, the Beaufort scale has lost entirely its original significance. The scale, instead of passing into oblivion together with the ships that it was designed for, has become rejuvenated, and is now the British standard for estimating wind force on land and sea all over the world.

For many years after the Tay Bridge disaster, experts in meteorological work were endeavouring to find what wind velocities corresponded to the numbers of the Beaufort scale. The Director of the Meteorological Office in his report dated 1906 states that " the gradual disappearance of the means of using Admiral Beaufort's specification has left the determination of the appropriate number for the description of the wind at sea to a tradition, which is apparently sufficiently well understood among sailors for practical purposes, but which has not been expressed in words. At the same time, the necessity for co-ordinating observations on sea with those on land makes it desirable to have the scale specified, if possible, and the improvements in the methods and practice of recording wind velocity, point to velocity as the measurement to which existing estimates should be referred." And as engineers and architects it is velocity, and its resultant pressure, with which we are concerned.

The work of ascertaining what velocities corresponded to the numbers 1 to 12, used in the Beaufort scale was enormous. One investigator, Mr. Curtis, alone dealt with 22,000 observations. The result was a new specification for estimating wind on land and at sea.

We are not concerned with wind at sea ; the land part of the specification is as given in Table 7.

In the report of the Meteorological Committee for the year 1906 it is stated that : during the investigation into the Beaufort scale it was found that when " the velocities arrived at empirically, as the appropriate equivalents of the Beaufort numbers, were plotted upon a diagram and a working curve drawn to represent them, the curve so drawn turns out to be very nearly identical with that represented by the algebraical equation $V = 1{\cdot}87\sqrt{B^3}$, where B is the Beaufort number and V is the hourly wind velocity in statute miles per hour. Adopting Mr. Dines' factor 0·003 for obtaining the wind

pressure in pounds upon a square foot, it follows that the relation between the Beaufort numbers and the wind pressure, P, is expressed by the comparatively simple relation—

$$P = 0{\cdot}0105\ B^3.$$

TABLE 7.

Beaufort Number.	Description of Wind.	Specification of Beaufort's Scale for Use on Land.	Mean Wind Force in lbs. per sq. ft. at Standard Density. $P = 0{\cdot}0105\ B^3$.	Velocity in miles per hour. $V = 1{\cdot}87\ \sqrt{B^3}$.	Probable Maximum Velocity attained by Wind in miles per hour.
1	2	3	4	5	6
				Averages.	
0	—	Calm ; smoke rises vertically	0	0	1·5
1	Light breeze	Direction of wind shown by smoke drift ; but not by wind vanes	0·1	2	4
2		Wind felt on face ; leaves rustle ; ordinary vane moved by wind	0·8	5	9·5
3		Leaves and small twigs in constant motion ; wind extends light flag	0·28	10	15
4	Moderate breeze	Raises dust and loose paper ; small branches are moved	0·67	15	24
5		Small trees in leaf begin to sway ; wavelets form on inland waters	1·31	21	30
6	Strong wind	Large branches in motion ; whistling heard in telegraph wires ; umbrellas used with difficulty	2·3	27	28
7		Whole trees in motion ; inconvenience felt when walking against wind ; umbrellas discarded in exposed places	3·6	35	46·5
8	Gale forces	Breaks twigs off trees ; generally impedes progress	5·4	42	56
9		Slight structural damage occurs (chimney-pots and slates removed)	7·7	50	66
10	Storm forces	Seldom experienced inland ; trees uprooted ; considerable structural damage occurs	10·5	59	78
11		Very rarely experienced ; accompanied by widespread damage	14	68	—
12	Hurricane	——	Above 17	Above 75	—

"Thus, if all the steps in this intricate investigation have been accurately followed, there appears to be underlying Admiral Beaufort's original specification the simple relation that the numbers selected are proportional to the cube root of the wind pressure."

These formulæ were used in the calculations for the values shown in columns 4 and 5 of Table 7, and afford an

easy means of converting the records regularly issued by the Meteorological Office into pressures.

The question of the relation of wind velocity to wind pressure can now be considered.

The formula already quoted in this paper is $P = KV^2$, where P equals the pressure per sq. ft., V the velocity of the wind in miles per hour, and K a constant.

It is the value to be taken for K that has caused a vast amount of discussion and research in recent years.

The value for K given in many text-books of the older sort is 0·005; this figure was attributed for many years to Smeaton, but when it was traced back it was found that it originated with a Mr. Rouse, and that Smeaton had quoted it in 1759.

Mr. Whipple, at the Royal Meteorological Society in 1888 said, "respecting this factor of 0·005, which was usually given for deducing wind pressure, from the velocity, he noticed that in the *Engineers' Pocket Book* for 1854 the factor was given as 0·00228, seeming to show that the correctness of the factor commonly given was then doubted." The latter value seemed strangely familiar, and on looking up the information given in a recent edition of a well-known *Engineers' Pocket Book*, it was found the value given for K when V was in feet per second was 0·002288, and when V was in miles per hour K was given as 0·00492. Apparently the 1854 book had become somewhat mixed up in the units; the other book quoted requires drastic revision on this particular point.

For instance, if we take a 68 miles per hour wind and K as 0·003, the pressure is 14 lbs. per sq. ft., but if we take K as 0·00492 the pressure becomes 22·75 lbs. per sq. ft., which is a difference of 8·75 lbs. per sq. ft., a serious difference when designs are being considered. Many other values for K have been obtained from time to time, but the one which stands out and is shown by recent experiment to be correct, is the 0·003 obtained by Dr. Unwin many years ago.

Of the experiments that have been undertaken by experts in this subject, the following results can be taken as the best known :—

Dines	$P = 0{\cdot}003\ V^2$
Froude	$P = 0{\cdot}00366\ V^2$
Langley	$P = 0{\cdot}00326\ V^2$ to $0{\cdot}0039\ V^2$
Stanton	$P = 0{\cdot}0032\ V^2$

whilst Dr. Stanton obtained a value of 0·0027 for K from his experiments with a uniform current of air artificially produced.

It can now be accepted that for flat solid surfaces placed at right angles to the wind the value for K is 0·003 or 0·0032, and Dr. Stanton considers that there is no reason to doubt that his constant holds good for all ranges of velocity up to the strongest gales.

The value for K obtained by Dr. Stanton is the result of an elaborate series of experiments carried out at the National Physical Laboratory, and fully described by him in the *M.P.I.C.E.*, vols. clvi. and clxxi.

The first part of the experiments was to expose plates and small models in a uniform current of air drawn through a channel 2-ft. diameter by a fan, and by very delicate measurements to determine the velocity of the current and its pressure on the windward and leeward sides of the plates and models. The second section of the experiments was to measure the velocity of the wind and pressure on boards and a lattice girder placed on a tower 50 ft. high erected in the grounds of the Laboratory. The results obtained in the artificial current of air and those obtained from the wind in the open air were then compared, and it was found that the difference in the value for K was due to an increase of suction, or leeward pressure, in the open air experiments.

The size of the pressure boards and value for K for each were :—

5 ft. by 5 ft.	$K = 0{\cdot}00320$
10 ft. „ 5 ft.	$K = 0{\cdot}00318$
10 ft. „ 10 ft.	$K = 0{\cdot}00322$

The conclusions arrived at were that "there appeared to be good reasons for supposing that the mean intensity of pressure on similar surfaces of areas greater than 1 sq. ft., exposed to the wind, is independent of their actual dimensions."

The presence of windward and leeward pressures on experimental boards and structures has been known for many years, and many experimenters and writers have dealt with it. The windward pressure is of course caused by the wind striking on a surface exposed to it. The leeward pressure appears to be caused by the flow of air round the edges of the obstacle, tending to draw some of the still air, behind, with it, and causing a partial vacuum. The direct pressure and the suction of the vacuum must be added together in order to arrive at what is known as the total pressure.

Interesting experiments to obtain the distribution of air pressure were made by Dr. Stanton in a uniform current of air on small models representing closed buildings with roofs at 30°, 45°, and 60° to the horizontal. These may be briefly summarized as follows :—

Model with 30° *roof.*—Plus pressure on the windward vertical wall and over about the middle third of the slope of the roof on the windward side.

Minus pressure over the remainder of the windward roof slope, and on the leeward slope and wall.

Models with 45° *and* 60° *roofs.*—Plus pressure on the windward vertical wall and slope, except at the ridge where it becomes minus.

Minus pressure on the roof and wall of the leeward side.

Whilst these results were obtained with an artificial current of air, Dr. Stanton says that "the action of wind is no doubt more complicated than that of a uniform current of air ; yet it seems probable that in displacements of considerable intensity, such as gales, the conditions of a uniform current may be approximately fulfilled, so that the distribution of the wind pressure on exposed structures may be regarded as that due to a uniform current."

Further experiments were carried out on a model roof with slopes of 56 sq. ft. in area, and so arranged that the angle of the slopes could be varied.

Values for K for use in the formula $P = K V^2$, where P equals the normal pressure on the roof in lbs. per sq. ft., and V equals the velocity of the wind in miles per hour, were obtained, so that it is only necessary to know the maximum velocity of the wind at any site, in order to arrive at the pressures on a proposed building at the site.

Dr. Stanton gives the result as follows :—

(*a*) Roof mounted on columns through which the wind can pass—

	Values of K_n.		
Angle of roof ...	30°	45°	60°
Windward side ...	+0·0015	+0·0028	+0·0034
Leeward side ...	negligible		

(*b*) Roofs of buildings in which the pressure on the interior may be affected by the wind—

	Values of K_n.		
Angle of roof ...	30°	45°	60°
Windward side ...	+0·0015	+0·0028	+0·0034
Leeward side ...	−0·0022	—	−0·0032

This case is equivalent to a building with openings on the windward side and closed entirely on the leeward side.

Comparing Dr. Unwin's formula which has been, and is, generally used for designing, but which does not take into consideration the leeward wind pressure, the following results are obtained :—

P_n = the normal pressure on the surface of a 30° roof, lbs. per sq. ft.
P = horizontal wind pressure.
K = constant for pressure on vertical face.
K_n = constant for normal pressure on 30° roof.
V = velocity of horizontal wind in miles per hour.
A = angle of roof = 30°.

Taking 30 lbs. per sq. ft. for vertical pressure of horizontal wind—

$$P = K V^2,$$
$$V = \sqrt{\frac{P}{K}} = \sqrt{\frac{30}{0{\cdot}003}} = 100 \text{ miles per hour.}$$

By Dr. Unwin's formula—

$$P_n = P \sin A^{1{\cdot}84 \cos A - 1},$$
$$P_n = 30 \times 0{\cdot}66 = 19{\cdot}8 \text{ lbs. per sq. ft.}$$

Dr. Stanton's formula—

Windward side $P_n = K_n V^2 = 0{\cdot}0015 \times 100^2 = 15$ lbs. per sq. ft.,
Leeward side $P_n = K_n V^2 = 0{\cdot}0022 \times 100^2 = 22$ lbs. per sq. ft.

Up to the present, assumed wind pressures, which are undoubtedly excessive, have been in general use for designing and very safe structures have been the outcome ; but with the advance of knowledge, the very heavy structures, now common, will gradually give place to more scientific designs, and a corresponding saving of material. New methods now being evolved will then become of the greatest importance.

Another phase of the investigations at the National Physical Laboratory was an endeavour to throw light on the much vexed question of the shielding effect of members of a lattice girder placed behind each other, and the effect of a windward girder on a leeward girder of a bridge.

From the small models it was found that with two circular plates placed $1\frac{1}{2}$ times their diameter apart, the total pressure was "less than 75 per cent. of the resistance of a single plate."

On increasing the distance to "approximately 2·15 diameters, the total pressure was equal to that on a single plate."

Again increasing the distance to "5 diameters, the total pressure was only 1·78 times that on a single plate."

"Experiments on square and rectangular plates gave corresponding results, it being found that the shielding effect of long rectangles was considerably less than in the case of circular plates, but in all cases the maximum shielding effect was observed when the plates were at a distance apart of approximately 1·5 times the least cross dimension."

Investigations were made on model lattice girders subjected to the uniform current of air. The "experiments were made on a single girder, and on two parallel girders with, and without a roadway, normal and inclined to the direction of the current. The shielding effect of the windward girder is very considerable, for both normal and inclined currents. When the two girders are connected by a roadway and are at a distance apart equal to the depth of the girder, the pressure on the leeward girder is 15 per cent. of that on the windward girder, and at twice this distance the pressure is 25 per cent. of that on the windward girder. The effect of the roadway appears to diminish the total pressure on the girders. When the direction of the current is inclined to the plane of the girders the resultant normal pressure is increased for small angles of obliquity, but not to any great extent, the value of the normal pressure when the current makes an angle of 75° with the plane of the girder, being approximately 5 per cent. greater than that for normal incidence."

The sheltering effect of the floor of a bridge has always been recognized. Claxton Fidler states in his treatise on bridge construction, "in the majority of cases we shall have a pair of girders united at the lower edge by a continuous flooring—in such cases it is sometimes assumed that the pressure on the whole structure is augmented by the presence of the horizontal floor. But it is very doubtful whether the skin friction upon the floor is not partially compensated, or neutralized, or more than neutralized, by the protecting effect."

Dr. Stanton also investigated the effect of wind pressure on a lattice girder 29 ft. long by 3 ft. $7\frac{1}{2}$ in. deep, the

total area exposed to the wind being 56·3 sq. ft. As the result of 200 observations it was found that the value for K was 0·00405.

If we compare this girder with a solid plate of the same overall dimensions and take the velocity of the wind at 68 miles per hour and also take the respective values for K we find—

	Area in sq. ft.	K.	V.	P.	Total Pressure.
Girder ...	56·3	0·00405	68	18·727	1054·33 lbs.
Plate ...	104·98	0·0032	68	12·819	1345·73 ,,

That is, the pressure per sq. ft. on the girder is 1·46 times that on the plate, whilst the total pressure on the plate is 1·27 times that on the girder.

It will be noticed that the area of the girder is roughly half that of the plate ; it may be that for different ratios of areas, other values for K would result and of course the total pressure would be varied.

The regulations relating to wind pressure on bridges have already been mentioned. In the L.C.C. (General Powers) Act, 1909, we find that : "For a roof the plane of which inclines upwards at a greater angle than twenty degrees with the horizontal the superimposed load (which shall for this purpose be deemed to include wind pressure) shall be estimated at twenty-eight pounds per square foot of sloping surface." Also, "All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than thirty pounds per square foot of the upper two-thirds of the surface of such building exposed to wind pressure."

Turning to the proposed L.C.C. Regulations for Reinforced Concrete—which may some day come into force—it is stated, "For a roof the plane of which inclines upwards at a greater angle than twenty degrees with the horizontal, the superimposed load, which shall for this purpose be deemed to include wind pressure and weight of snow and ice, shall be estimated at twenty-eight pounds per square foot of sloping surface on either side of such roof.

"All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than twenty pounds per square foot of the whole projected surface normal to the direction of the wind.

"All structures or attachments whatsoever in connection with a building, including towers or other parts which extend above the roof flat or gutter adjoining thereto, shall

be so designed as to resist safely a wind pressure in any horizontal direction of not less than 40 lbs. per sq. ft. of the whole projected surface normal to the direction of the wind."

It is impossible to include the regulations of the foreign nations, and at the same time keep this paper within respectable limits, but those for the chief towns of the United States are of especial interest, because of the great height of some of the buildings.

For instance, the largest office building in the world, the Equitable, at New York, is a building 542 ft. high from the street-level, whose steel framework weighs 32,000 tons. The Woolworth, also at New York, is the highest office building in the world : 760½ ft. from the street-level to base of flag-pole on the tower, whose heaviest basement columns carry on each 4,740 tons, of which 1,300 tons is due to the action of the wind ; whilst the weight of the steel framework is 24,100 tons. Probably the only commercial building in England approaching these dimensions is the Royal Liver building at Liverpool, which rises to a height of 290 ft. above street-level, and is built entirely of reinforced concrete. To compare with these buildings we have St. Paul's Cathedral, about the height of which there seems to be some doubt, judging from recent technical papers. From the floor of the cathedral to the top of the cross it seems to be more or less 360 ft.

The principal wind regulations for the United States are summarized in Table 8.

It is interesting to note that Chicago, "the windy city," and San Francisco, which is in an exposed position on the Pacific Coast, specify the lowest wind pressures.

As the title of this paper suggests, it is the result of notes accumulated during many years, revised, and brought up to date and now strung together in a more or less disconnected manner.

It was originally intended to deal also with the result of wind pressure on steel and reinforced concrete structures, but this part of the subject must be left to the future, although it is a matter of general interest and also of interest to such training centres as the Northern Polytechnic and L.C.C. School of Building, where much good work in design is being done.

The amount of published matter on this subject is considerable. An endeavour has been made to embody within this paper the reliable information that is of practical use.

Many authorities are mentioned in this paper. The

Journals of the Royal Meteorological Society, the minutes of the Institution of Civil Engineers and kindred Societies, engineering and other technical papers, Government publications, etc., have all been drawn upon. I am greatly indebted to Dr. W. N. Shaw, F.R.S., Director of the

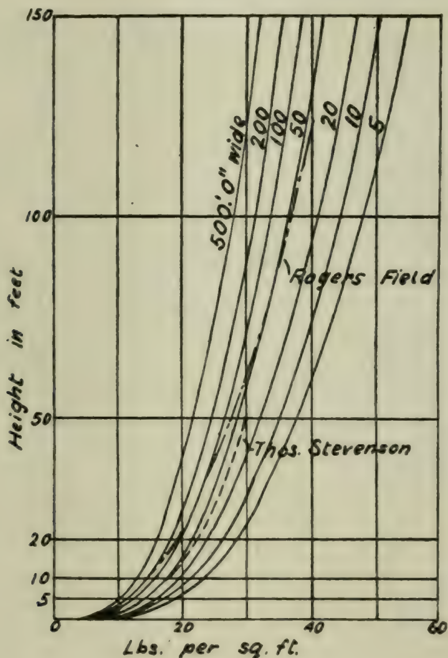
TABLE 8.

Town.	Horizontal Wind Pressure in lbs. per sq. ft.		Allowable Increase of Stress in Wind Bracing and Members Subjected to Wind Loads.	Remarks
	Exposed Position.	Sheltered Position.		
New York... ... Rochester ... ... Minneapolis ... Louisville ... ... Atlanta ... ... Newark ... ... Jersey City ...	30	30	50 %	Except those under 100 ft. in height in which the height does not exceed four times the average width of base.
Philadelphia ... Syracuse ...	30	25 to 35	30 % 25 %	25 lbs. at tenth story less $2\frac{1}{2}$ lbs. for each story lower and $2\frac{1}{2}$ lbs. additional for each story above to a maximum of 35 lbs.
Detroit	30	30	25 %	—
Baltimore ... ... Buffalo	30	30	none	—
Chicago ... ... San Francisco ...	20	20	50 %	—
St. Louis	30	30	20 %	—
Seattle	None	specified	50 %	—
Boston	Provision	for wind	bracing sha	ll be made where [necessary
Providence ... Los Angeles ...	No regu	lations		

Meteorological Office, for much information given in this paper, and also for the loan of lantern slides; and to Mr. H. W. Richards, Principal, L.C.C. School of Building, for releasing me from my engagement there to-night. Also to Messrs. Negretti & Zambra for the exhibit of their wind-measuring instruments.

CORRESPONDENCE.

PROFESSOR HENRY ADAMS, M.Inst.C.E., etc. (President), wrote that he much regretted being prevented from attending the meeting, being called away to Bury, to investigate a reinforced concrete failure, which had occurred there. He has for many years taken a great interest in questions of wind pressure. The accompanying diagram may prove acceptable to



the meeting; the formula is log $p = 1{\cdot}125 + 0{\cdot}32$ log $g - 0{\cdot}12$ log l, where p=the ultimate wind pressure in lbs. per sq. ft. necessary to be allowed for against a plane surface perpendicular to the direction of the wind; g=height of centre of gravity of surface, considered above the ground-level in feet; l=width in feet of the part to be taken as one surface. He shews the result of his formula by graphic curves.

and has added the curves given by two other engineers. He added, that he believed Mr. Etchells to have a formula also for the variation of pressure with the height.

MR. JOHN W. GRANT, A.M.C.I., A.F.P.W.Inst. (Lond.), wrote :—Perhaps you will permit me to add a few notes on wind pressure, which I have just received from Mr. Donald MacLean, F.R.A.S., of the Paisley Observatory, Scotland.

The greatest storm, recorded at the Coats' Observatory since it was founded in 1882, was a momentary gust of wind, travelling at the rate of 98 miles per hour and the pressure 29 lbs. per square foot, at 9.5 a.m. on December 22, 1894, registered from their Richard anemometer. Another storm, recorded December 24, 1912, at this Observatory, when the velocity was 97 miles per hour at 8.55 p.m., and the pressure was then 28½ lb. to the square foot. Registered by a pressure-tube recorder.

In Glasgow on the same day the tide rose 6 ft. above H.W.O.S.T. Wind velocity 73 miles per hour.

The leading observatories in Scotland are those at Blackford Hill, Edinburgh ; Dowanhill, Glasgow ; Oakshaw Hill, Paisley ; and at the new Government Institution at Eskdalemuir, Dumfriesshire.

Mr. Charles R. Gibson, F.R.S.E., writing on the equipment of an observatory, states that at Paisley on one occasion "the hurricane was so great that it tore the 'windmill' to pieces and carried its parts to different quarters of the town." The windmill is made of aluminium and is delicately poised, so that it is set in motion by a very gentle breeze.

The late Professor Grant, Astronomer of Glasgow University, took a great interest in Coats' Observatory, Paisley, and Mr. MacLean was one of his assistants.

DISCUSSION

MR. EWART S. ANDREWS, B.Sc.(Lond.), M.C.I. :—There is one point that I want to make before I forget it, and that is with regard to the high records which Mr. Keevill refers to as erroneous. I want to keep in mind the point that these wind-

caused forces are in the nature of dynamic forces. We ought therefore to be very careful not to eliminate the dynamic effect of them and merely measure them as static pressures ; when we speak of designing for a pressure of 30 lbs. per square foot, that is really only equivalent to 15 lbs. per square foot static pressure. I don't know whether Mr. Keevill can give me some information upon the wind pressure in streets and in valleys. It seems to me, that the effect of converging currents of air down a street, and in comparatively open country near the sea the effect of a slight sloping down, have the effect of giving an apparently tremendous increase in the wind pressure ; but I have no anemometer records of that. Also it has an effect, which is rather important to us as designers. You very often get the wind pressure acting as a downward pressure, as people who live in country near the sea discover to their discomfort when they try to light the fire. Mr. Keevill makes an eloquent appeal for the reduction of wind pressure, and in that I strongly support him, if the information derived from scientific observation is that the wind pressures, which we take, are too high. I think we want to avoid a condition of mind, that because a certain wind pressure is specified in certain regulations that is the wind pressure, which exists or which will be experienced. The Legislature and the statutory laws have a sort of crystallizing effect upon our minds and tend to make us accept things as gospel, because we find them in print ; it is very desirable, that we should discover exactly what is the maximum wind pressure, likely to be experienced in any locality ; and, if, as I believe, the evidence shews it is considerably less than the amount specified, I think action should be taken to give relief in that respect. What I strongly urge is this, that if we are going to make any allowance at all we should not do it in a roundabout way, which seems to be dear to the hearts of legislators, such as the method of allowing a higher working stress. That reminds me of allowing a variable factor of safety for columns, and I think it is extremely unscientific and roundabout. If the wind pressure is less than the amount specified, instead of putting up your stress, put down your wind pressure,

and let us have it quite clear and straightforward. I would like to draw the attention of the meeting to one paper, which Mr. Keevil has not referred to and which contains, so far as I know, most information on the distribution of wind pressure on roofs and upon the suctional effect of the pitch of the roof and the high walls of a building. It is a paper by Albert Smith, called "Wind Pressure on Buildings." It occurs in the *Proceedings of the Western Society of Engineers* for 1912.

Mr. Percy J. Waldram, F.S.I., Licentiate R.I.B.A., M.C.I., and Mr. W. Cyril Cocking, M.C.I., then spoke.

Mr. S. Bylander, Past Chairman J.I.E., M.C.I.: —One thing that interests me most particularly about wind is the provision required to resist it in a structure. When one is faced with a problem of designing a building, then I think one realizes more intimately what wind really means. I had to design a tower some long time ago, and in taking the moment about the base of the tower it is there, as a previous speaker pointed out, that we find an enormous stress and an enormous moment. I certainly think that the pressure, which is usually assumed, that is, 30 to 40 lbs. per square foot—is on the high side, particularly as all buildings are not exposed to such extreme winds. I further think, that the stress due to wind should not be added to the stress due to dead load or to statical load. I think that a higher total stress should be allowed in the case of designing structures to resist wind. When calculating the brackets or the knee braces required to resist the wind, one finds that one must make an important addition to the ordinary sections, which would otherwise be required. Supposing that our maximum stress is 16,000 lbs. for the load specified, I think that if the stress due to wind is added you should be allowed to increase that stress by at least 20 per cent. It is very interesting to study structures which have been put up and which have stood thirty or forty years, and to calculate the stresses which would occur in those structures, had the full wind pressure been applied. I

took an interest in this question some eight years ago. I had to deal with a new building of great dimensions, and was told that they wanted the same as the one they had had before. I calculated the stresses, and really I would not like to have taken the responsibility for such a building. I found that the building, which had stood for thirty or forty years, could only resist a wind of from 5 to 6 lbs. per square foot, assuming that the stresses would not exceed the usually accepted stresses. You will understand, that I do not say that the wind never was more than 5 lbs., but I *do* say, that if the stresses in the materials were not excessive, the wind could not have been more than 5 to 6 lbs. per square foot. What actually happens is that the materials may be stressed to nearly the breaking-point on some occasions. I heard Mr. Cocking suggest, that the roof truss should be designed, so that it would carry its load whether upside down or the right way up, but I take it that he speaks more forcibly than he really means. I quite agree with him, that the members should be stiff sections. I think it is bad practice to use flat bars, instead of angles. I have seen time after time how these pieces are erected. They are not straight, and if the load is applied, the different members do not take the proper share of the load.

MR. E. FIANDER ETCHELLS, Assoc.M.Inst.C.E., A.M.I.Mech.E., F.Phys.Soc. (Member of Council C.I.), made the following remarks :—

INSTANCES OF HIGH PRESSURE.

I have one or two instances of high wind pressures to add to those given by the author. The first is 27 lbs. per square foot at Greenwich on December 3, 1909. That figure was given by the Astronomer Royal, and was based on the assumption that the velocity of the air was three times that of the cups of the Robinson anemometer. If we take the ratio of velocities as being two instead of three, I estimate that the pressure would only be 12 lbs. per square foot instead of 27. A wind pressure of 31·5 lbs. was,

however, recorded at Pendennis Castle, Falmouth. That was the highest gust, registered during the last fifteen years. An examination of the records of the Royal Meterological Office shows that a pressure of 23 lbs. was the highest mean velocity sustained for an hour. That was at Fleetwood. It is certainly very remarkable that the mean pressure sustained for an hour, and not a gust, was 23 lbs. per square foot.

Practical Examples.

The Forth Bridge was designed for 56 lbs. per square foot, but that was to meet the Board of Trade requirement. The structure is not specially exempted from the risk of corrosion, and it is stated that the painters are never off the bridge. In times to come, when some inevitable corrosion has taken place, it will still be able to resist the pressures that actually do occur at that period, and with a sufficient margin of safety, although at present the safety factor is admittedly much greater than the limits of immediate necessity. Another building, that has stood a reasonable test of time was designed to carry safely a wind pressure of 25 lbs. per square foot. I refer to the Crystal Palace, at Sydenham ; but as the framing is of cast iron, there is less liability to corrosion. At the Palace there has been considerable damage to glass on account of the wind, but the windows have frequently blown outwards, on the leeward side. This is an indication, that in the geographical conditions there the leeward pressure (the drag, or suction) is as great as the positive pressure on parts of the windward side. It is also stated that one may easily stand on the windward side facing the direction of the wind in the front of the building. The wind is directed, apparently, over the roof, and does not come into the angle formed by the building and the ground.

London Building Acts.

In regard to legislation, in the latest piece of legislation in London the wind pressure is not the 30 lbs. per square foot referred to, but is reduced

to 20 lbs., and the 20 lbs. is for buildings generally. For the relatively small area of chimneys and towers, which stand above the roof, it is certainly 40 lbs., but the cost of the wind bracing lies in the bracing of the whole building generally, and not the isolated turrets or chimneys, standing above the level of the roof. Wind pressure is a dynamic load which, as shown on the screen this evening, may instantaneously rise from zero to the pressure corresponding with the velocity of 66 miles per hour. A sudden gust is clearly a suddenly applied pressure, and we know that in the general consensus of engineering opinion you must reduce shock loads to equivalent static loads. The pressures given in the regulations referred to are not supposed to represent the actual dynamic or shock pressures, but are intended to be the equivalent static load to correspond thereto. During the period, which has elapsed since the passing of the "Steel-frame Act" some changes have been made—changes notably of a reduction of wind pressure, from 30 lbs. per square foot on the upper two-thirds to 20 lbs. per square foot for the whole area. That does not reduce the total amount of pressure over the whole building, but it does reduce the height of the centre of pressure.

Approximation to Unwin's Formula.

With regard to Professor Unwin's formula, expressing the results of Hutton's experiments on inclined surfaces—viz.—

$$p_n = p \sin \theta^{(1 \cdot 84 \cos \theta - 1)}$$

where p_n = *pressure normal* to the surface of the roof in pounds per square foot of sloping surface,

p = pressure of the wind in pounds per square foot i.e. the horizontal pressure against a vertical surface,

θ = slope of the roof in degrees, measured from the horizontal plane.

I submit the following approximate formulæ for your consideration :—

$$p_n = 0{\cdot}022\,\theta p$$

or

$$p_n = \frac{11}{500}\theta p$$

It should be noted that Unwin's rule involves the expression—

$$\sin\theta^{[(1{\cdot}84\cos\theta) - 1]}$$

whilst the approximate rule contains the expression—

$$\frac{11}{500}\theta$$

If either of these expressions exceeds unity, we should take—

$$p_n = p$$

The results of each method are compared in the following table :—

Slope in Degrees Measured from the Horizontal Plane.	Unwin's Formula Value of $p\sin\theta^{1{\cdot}84\cos\theta - 1}$.	Etchells' Approximate Rule Value of $0{\cdot}022\,\theta p$.
0	0	0
5	$0{\cdot}125p$	$0{\cdot}11p$
10	$0{\cdot}24p$	$0{\cdot}22p$
20	$0{\cdot}45p$	$0{\cdot}44p$
30	$0{\cdot}66p$	$0{\cdot}66p$
40	$0{\cdot}83p$	$0{\cdot}88p$
50	$0{\cdot}95p$	p
60	p	p
70	$1{\cdot}02p$	p
80	$1{\cdot}01p$	p
90	p	p

The total force could be split into two factors for the windward and leeward effects.

Thus, let $\overline{\mathrm{W\,F}}$ represent the *windward factor* and let $\overline{\mathrm{L\,F}}$ represent the *leeward factor*, then—

$$p_n = \overline{\mathrm{W\,F}}\,\theta p + \overline{\mathrm{L\,F}}\,\theta p$$
$$= (\overline{\mathrm{W\,F}} + \overline{\mathrm{L\,F}})\,\theta p$$

In Unwin's formula, expressing the results of Hutton's experiments, these two factors were not separated.

We only know that $(\overline{\mathrm{W\,F}} + \overline{\mathrm{L\,F}}) = 0{\cdot}022$ approximately.

Let V = *velocity* of horizontal wind in miles per hour.

We can then combine the fomula—

$$p_n = \frac{22}{1000}\,\theta p$$

and—

$$p = \frac{3}{1000}\,\mathrm{V}^2$$

Thus—

$$p_n = \frac{22}{1000}\,\theta \,.\, \frac{3}{1000}\,\mathrm{V}^2$$
$$= \frac{66}{10^6}\,.\,\theta\,\mathrm{V}^2$$
$$= 0{\cdot}000066\,\theta\,\mathrm{V}^2$$

This is the general form, but if we take a constant slope of 30°, this will reduce down to

$$p_n = 0{\cdot}001980\,\mathrm{V}^2$$

Now, this is Hutton's and Unwin's result expressed in the form of Stanton's equations.

Let us take a particular case. Let V = 100 miles per hour.

In this case—

$$p_n = 0{\cdot}001980\ V^2 \text{ becomes}$$

$$= 0{\cdot}00198 \times 100^2$$

$$= 19{\cdot}8 \text{ lbs. per square foot}$$

This is identical with the results given by the author, with which it should be compared.

Effect of Wind on Structures.

With regard to the failure of roofs under wind, I have seen several roofs which have been lifted by the wind and sometimes overturned and carried away, but I have not seen a roof crushed in or broken in by the wind. I have seen roofs four hundred years old, that have been bent under the dead load of tiles, but I have not seen a roof absolutely broken in by the wind from outside. I think the reason is, that in the past we designed our roofs to resist the vertical or outside loads which come upon them, and therefore they stand that, but we have not designed them to resist uplifting pressures from the inside, or suctions, as it were, outwards. In all roofs that are very much open to the air it seems desirable, that flat bars should not be used.

Chimneys.

One of the previous speakers expressed the opinion that if the wind pressure were 20 lbs. per square foot half the chimneys in London would be demolished. I have examined a great many chimneys, and many of them have been those where stability has been called into question. I have found, that it would take a wind pressure varying from 40 to 60 lbs. per square foot to overturn the ordinary house chimney designed in accordance with the proportions set out in the London Building Act, 1894. In other words, the old requirements of 1844, 1855, and 1894 for chimneys have the effect of setting the standard of stability, which corresponds with an ultimate resistance to overturning of from 40 to 60 lbs. But some

of those chimneys have been overturned. In some of these cases we are able to trace it to bad mortar, or weak joints, or faulty workmanship, or cracks, or dislodgements. Another speaker asked, that in steel-frame buildings they should be allowed an extra stress when they include wind pressure in the calculations. He asked for 20 per cent. The fact is you are allowed 25 per cent. in the pillars, at any rate, and the bressumers being strutted by the floors are in quite a good case.

MR. E. GOLD, of the Meteorological Office :—I do not know if it is permissible for a visitor and a stranger to intervene in this very interesting debate. My particular business is concerned with meteorological matters, and I am not in any way expert in concrete building, but I have listened with a great deal of interest to the debate this evening. In these stormy times it is something like entering a haven of calm to hear wind being discussed in connection with something which is not a vessel of war, but so peaceful a subject as the safe building of places for people to work in. Mr. Keevill gave a very good account of the history of the development of the experiments on wind pressure, but he only touched very briefly on what is to some of us one of the most romantic developments of modern times, and that is the calculation of the wind velocity from the pressure distribution. The word "pressure" has been used in so many different senses to-night, that one perhaps must explain what pressure distribution means. On the last chart, which we saw on the screen, was a set of lines called isobars, along each of which the pressure of the atmosphere, as measured by a barometer, has the same value, but a value different from that belonging to the preceding and succeeding lines. It is possible nowadays for meteorologists to calculate effectively, from the lines which you saw on the board, what the actual velocity of the wind on any given occasion is likely to be at a height of 1,500 ft. or so. If those lines are about 50 miles apart, you will get, roughly, a velocity of 80 miles an hour, and that can be calculated with a great deal of accuracy. But that, of course, is really outside the

region of the construction of buildings, at any rate in this country, although it may verge on the structures, which I had the pleasure of seeing a few months ago in New York, the gigantic Equitable and Woolworth buildings. There is one point, which struck me as being of some importance in connection with the application of wind pressure to buildings, and that is the effect of snow or rain falling in the wind. The density of atmosphere is only about one-thousandth of the density of water, so that if you had two raindrops a millimetre in diameter in every cubic centimetre of the air, you would, roughly, be doubling the density of the air, and as you know that rain, and more especially snow, is carried along with the wind, it seemed to me that it might possibly explain some of the differences between pressure-plate experiments and the wind, which one gets from the Robinson or pressure-tube anemometers, because on the pressure plate you would get not only the wind, the moving object, but the driven snow ; and although I do not know the relative density of air in which falling rain or snow is included, it is quite conceivable, that it might be nearly double the normal density. In that case you would be getting on the pressure plate double the pressure recorded on the wind instruments of the usual type. In the Robinson it is a matter of pure velocity, and in the Dines instrument it is a matter of air pressure, and if the snow pressure did anything at all, it would block up the tube and give no result at all. So much for that. It was brought to my mind, because the greatest wind velocity ever experienced in London occurred during the great snowstorm in January 1881, when—I was not in London myself at the time—I believe Londoners suffered very severely from having their milk supply cut off. There was an easterly blizzard for two days. The wind velocity was, in the mean taken over an hour with the proper factor, 43 miles an hour, which has never been exceeded since. The nearest approach to it in recent years was in March 1913, when the mean velocity reached something like 40 miles an hour ; but those two occasions and one other are the only occasions in the last forty years, in which a gale has blown in London—that is, a gale as defined

in the specification of the Beaufort scale given in the paper. There has been some discussion as to the applicability of the records from the coast stations to buildings in an inland position. Of course, it is quite clear, that it is quite unreasonable to take the record of a coast station and say, "We will act in an inland position on that hypothesis." It recalls to me very forcibly an occasion on which I had to give evidence in regard to one of the gales in London, and the judge and the jury appeared to regard the matter as of very little importance, because velocities of 30 or 40 miles an hour greater had been registered at coast stations. What is an act of God in London is a normal (not everyday, but every year) effect at exposed coast stations, not only in Scilly and Pendennis, but at Quilty, on the west coast of Ireland, where velocities of 98 miles an hour have been recorded. If you had a velocity of 98 miles an hour in London, it would be an act of God ; it could not occur in the normal course. It has not occurred in the last fifty years, and it would not occur, unless you had something similar to the tornado in South Wales recently. With regard to the outer wall of a room exposed to the wind, it would be very interesting to know exactly how much opening you have to leave in your room, to get a real suction effect inside it. At the Observatory at Ben Nevis the barometer used to pump up and down very considerably—that is, the pressure in the room was varying all the time, owing to the wind blowing over the top of the chimney. As the wind varied, the pressure in the room varied, I believe on some occasions by as much as half an inch. So that on occasions there must be, even in what is comparatively a closed room, a very considerable suction effect, and it is a matter probably for direct calculation to know, how big the ingress must be to produce an effective suction.

In regard to the formula for the pressure constant, I had the impression that the constant 0·005 (which used to be used) was computed on the hypothesis, that the obstacle simply stopped the motion of the air ; that it depended upon the rate of destruction of momentum on the assumption, that it was a simple matter of particles coming up and having their

momentum destroyed. No doubt Mr. Keevill will correct me if I am wrong, but I had that impression, that it was purely theoretical deduction based upon that hypothesis. There are one or two points in the paper, to which I should like to make reference. The author suggests (on the first page) that gravity is a possible cause of the motion of the air. Of course, gravity does vary over the surface of the earth, but the earth is so constructed, and the atmosphere would take up such a position, that if there were no other factor gravity would keep it all in equilibrium. The form of the surface of the earth is such, that the variation of gravity over it is just what is necessary, to counteract the effect arising from the rotation of the earth. I make these suggestions, not purely in spirit of criticism, but partly because I have a confession to make in one minute. Mr. Keevill speaks of Dines' anemometer and connects it with the anemo-biagraph made by Negretti and Zambra. He showed a picture on the board, and you would probably see at once that there was a difference between the picture and the instrument on the table. The float in the picture, which acts as the recording part of the apparatus has a special shape, and it is simply by the special shape of the float in the water, that the pressure is converted into linear increments of velocity. But in the instruments made by Messrs. Negretti and Zambra this effect is produced by springs. Speaking from my own point of view, I should say, that the special shape of the float, which cannot change, is preferable to springs, and as a matter of fact at the stations connected with the Meteorological Office the form without the springs is invariably used, although one or two records are received from volunteer stations, which use the Negretti and Zambra anemo-biagraph. I think Mr. Keevill might make it clear in his paper, because Dines' instrument is a little different from this one. On page 34 he gives a table showing velocities at South Kensington and at Kew. It shows very much smaller velocities at South Kensington than at Kew, and that is generally the case, but about a week ago I learned that the anemometer at South Kensington has been out of order for a short time. I ought to explain how it is possible

for an instrument to get out of order, without being immediately discovered. The tubes which convey the pressure and the suction from the top to the bottom may develop a leak or get some water in them, and that is one of the things, one has to be careful about, that there is no bend in the tube, otherwise water will condense in it. The suction part of the South Kensington apparatus has been out of order for some time, and part of that time is covered by those records. The table on page 34 ought therefore, I think, to be modified.

MR. R. GRAHAM KEEVILL :—I do not think there is very much for me to reply to, as apparently you have all replied to each other. Mr. Andrews mentioned wind pressures in streets and valleys. I am afraid I haven't any records for streets, but I believe that I have some notes about wind pressures in valleys. I do not know whether it would be any advantage, but I will see what I can do. For streets I do not think there are any available ; I should only be too glad to have them. With regard to the downward pressure of wind, there is a downward and an upward pressure to a certain extent by the movement of the atmosphere, but to what extent I am not quite sure. It depends on the state of affairs at the time the wind is blowing, but undoubtedly there is a certain amount of vertical pressure. Neither am I able to give the basis on which Professor Adams compiled his tables. I refer in the paper to them (of course I did not know that Professor Adams was not going to be here) specifically, with the idea that he would probably explain how they were obtained. I hope he will do so, perhaps in writing, in our TRANSACTIONS. In regard to the paper by Mr. Albert Smith, I knew that paper and others. My trouble in writing this paper has been to eliminate information. There is so much, that is valuable, that I could not possibly get it all in, and I have had to cut out much that I would have been only too glad to put in had I had the space. It is a very big subject, and it is not an easy matter to cover all the ground in a paper. One would rather write a book on it. Mr. Waldram says I have not made any new rules. I will leave

that to Mr. Etchells. I quite agree that the old idea of 56 lbs. per square foot is simply absurd. Only last week I saw the calculations of a designer, who ought to have known better, who was taking 56 lbs. per square foot for the design of a timber building. It is absurd, and I quite agree with Mr. Waldram, that the 56 lbs. pressure ought to be killed once and for all. With regard to the coefficient for K, Mr. Waldram generally has taken, he says, 0·0032; I have taken 0·003 in my paper throughout. That is the official value taken by the Meteorological Office, and that is the only reason why I have taken it. With regard to the question of stream-line action, I am afraid it is rather too late for me to deal with that now. The inertia of the Robinson anemometer, to which Mr. Waldram also referred, is hardly momentum; I may have expressed myself rather badly, but prefer inertia. With regard to Dr. Stanton's pressure of wind on closed structures, all his first experiments were on closed models, and I think the figures, Mr. Waldram has given are probably contained in Dr. Stanton's first paper, and I think that he recommends that the results for practical designing should be his figures for open on one side and closed on the other.

MR. WALDRAM :—I think his figures are for closed models on columns, through which the wind can pass harmlessly, for open models; and then he has some other figures for open roofs on a closed building.

MR. KEEVILL :—I think most of Mr. Cocking's remarks and Mr. Bylander's I can buckle together for the sake of saving time. They are essentially questions, with which I had not dealt in this paper. I have dealt with pressures, and with the exception of the few regulations at the end I have not discussed the actual stresses in a structure. It is a big subject, and the information, which has been given to-night, shows that a big discussion would arise here on a paper dealing with the results of wind pressure on buildings. Both Mr. Cocking and Mr. Bylander dealt with the practical part of it, with which, of course, I have every sympathy. With regard to the

question of skin friction on roofs, I do not think that the falling of the tile roof had anything to do with skin friction. The application of the London County Council regulations has been dealt with already by Mr. Etchells. As to roof trusses and all members having to stand compression, that is all very well where you have tension members large enough to put in angles or tees and conveniently get in the rivets. Otherwise you would get an excessively heavy roof.

Mr. COCKING :—I believe it is the standard practice in America to make all the members of roof trusses of such a section, that they can withstand compression.

Mr. KEEVILL :—Of large sizes?

Mr. BYLANDER :—Of any sizes.

Mr. KEEVILL :—That means, that you have small rivets in small roofs. Mr. Etchells quoted some figures for pressures, which doubtless he has satisfied himself are correct. The Astronomer Royal, when he gave evidence at the Tay Bridge inquiry, was rather incorrect; but Mr. Etchells has doubtless satisfied himself on that score.

Mr. ETCHELLS :—I am satisfied, that the Astronomer Royal should have said half the pressure that he gave. I estimated that Dr. Shaw under the same conditions would deduce a pressure of 14·5 lbs. per square foot, and not the 27 lbs. of the Astronomer Royal.

Mr. KEEVILL :—The sudden rise of a gust from zero to 58 miles an hour is quite exceptional; ordinary squalls do not rise to that vast extent. With regard to Dr. Unwin's formula, there are several ways of simplifying, and the simplest way is to look at a section book, where the constants are all worked out. The gentleman from the Meteorological Office, who has spoken, has asked me to explain how 0·005 was obtained as the value for K, but I must give it up at this late hour. With respect to the question of gravity in the first part of the paper, I qualified

it by saying "and possibly gravity." I did not say it *was* gravity.

MR. GOLD :—I tried to explain that it could not even be possibly gravity !

MR. KEEVILL :—We will call it impossible, then ; I am quite open to correction on a point like that, and I am very pleased to accept it. With regard to the Dines anemometer and Messrs. Negretti and Zambra's anemometer, I know there is a difference between them. Although I might refer to it as Dines' in the paper, it is quite plain that it is the other one. I must thank you very much for the careful consideration, that you have given to this windy subject and the information, that it has drawn forth. I am quite aware, that the discussion is probably more valuable than the paper itself.

FIFTY-NINTH ORDINARY GENERAL MEETING

Thursday, March 18, 1915

THE FIFTY-NINTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, March 18, 1915, at 7.30 p.m.,

Professor HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

The following applicants were elected :—

Membership.

Ernest Godfrey Page, A.R.I.B.A., Architect, 11 Gray's Inn Place, London.

Associate-Membership.

Ernest Edmund Larkin, Structural Steelwork and Reinforced Concrete Designer, 80 Goswell Road, London, E.C.

STUDENTSHIP.

ROBERT F. GALBRAITH, Assistant Builders' Foreman, 44 Longton Grove, Upper Sydenham, London, S.E.

MR. OSBORN C. HILLS, F.R.I.B.A., M.C.I., District Surveyor for the Strand, then read his paper, entitled—

THE LONDON BUILDING ACTS, WITH SOME SUGGESTED AMENDMENTS.

THIS paper is submitted with the view and the hope that a new Act will shortly be prepared embodying the existing Acts, By-laws, and Regulations. In 1894 our present London Building Act was passed, and thereby cancelled some thirteen existing Acts which had to be read together. Twenty-one years have elapsed since then; many decisions have been given in the High Courts, some of which completely annul the intention of the Act; many amendments have been framed, some of which have become law; and it will soon be necessary to repeat the process of 1894 and pass a new Act embodying all the existing laws and regulations, with certain desired modifications and improvements. Such improvements should be the carefully considered results of the various authorities: the Royal Institute of British Architects, the London Society, the Town Planning Association, architects, surveyors, structural engineers, builders, and property-owners should now unite in preparing such improvements in the Acts as they consider essential in readiness for submission to the proper authority. The London County Council "fathered" the Act of 1894 through Parliament, and the writer believes they have already collected much material for a proposed new Act.

It is not suggested that this Institute should take the initiative in the matter, but having already been recognized by Parliament in the 1909 Amendment Act

it should be prepared at the right time to suggest such amendments as will conduce towards sound construction on economical lines.

It is not proposed in this paper to deal exhaustively with the origin of Building Law, though the subject is one of extreme interest. Most of us will recall the recent remarks of Mr. Etchells on the Babylonian By-laws when he reminded us, *inter alia*, that the death penalty was inflicted upon the builder who (or whose men) should build so defectively as to endanger the stability of the fabric, and directly or indirectly cause loss of life.

The paper read by my colleague, Mr. Davidge, before the Royal Institute of British Architects as recently as April 20, 1914, mentioned that a builder who contravened the Building Law was to be publicly whipped until " his body be bloody."

So much for the builder of bygone times, but the architect fared no better. The late Professor Aitchison, R.A., President of the Royal Institute of British Architects (and some time District Surveyor for Wandsworth), was fond of quoting from the Classics, and could tell many good stories of his professional prototypes. I remember when the question of " fees " was under discussion he told me that Diocletian, upon the completion of a building, called for the architect (probably a Greek and a slave), and if he (Diocletian) were pleased the architect could go free ; if Diocletian were not pleased, the architect was crucified.

Coriolanus (489 B.C.) had occasion to enact regulations on concrete buildings ; for on his going one day, accompanied by a herculean general, to visit a sick soldier in his home the building collapsed. The herculean general, however, succeeded in holding it up while Coriolanus crawled out with his wounded soldier.

" Investigation will prove that the higher the standard of the profession in a country the greater the responsibility attached by law to the profession. This is as it should be, for it recognizes the importance of the subject, and, by inference, points to the necessity of some qualification being set. This is recognized from earliest times. In the earliest known code, that of the Babylonians, compiled by Hammurabi,

2250 B.C., provision is made for due compensation to be paid by the architect to the owner in the event of any disaster happening to the building during construction or after it is completed (see Edicts 238-233 inclusive). All our present laws on the subject are built more or less on the Edicts of Hammurabi, and the only inference we can draw is that the erection of a building has been considered at all times as an occupation of importance which should only be exercised by properly qualified individuals." *

The Building Act of the Emperor Zeno † is the earliest I can trace of which we really have a fairly complete knowledge.‡ It governed the elevations and distances of contiguous private buildings in Constantinople and was administered by building police. Though originally intended to control the buildings in Constantinople only, Justinian § afterwards enacted that it should be the universal law of the empire. The emperor assures the reader that he (Justinian) has taken pains to avoid all strictly legal or forensic expressions (used by Zeno), and by the substitution of descriptions intelligible to all, to make it easy for every one to understand of himself the meaning . . . without having recourse to the instruction of a technical expositor or commentator.

I fear Justinian would have disliked the very look of our new Reinforced Concrete Regulations.

Both Zeno and Justinian, however, were alive to the importance of sound construction, including the use of concrete, though "a view of the sea" was equally to be sought. It is interesting to find so far back as the sixth century that the ground landlord, the architect, and the contractor all are recognized, while the clerk of the works was probably the personage then called the superintendent. All (even the ground landlord) were liable to a fine of twelve pounds weight of gold for illegal construction

* Extract from paper read by Mr. J. S. Archibald on "A Statutory Qualification for Architects," at the Architects' Congress in London, 1906.

† Zeno reigned A.D. 474-491.

‡ See paper read at the Academy of Sciences, Berlin, February 8, 1844. translated by W. R. Hamilton, Esq., F.R.S., for the Museum of Classical Antiquities, October 1851 ; see R.I.B.A. Library.

§ Justinian reigned A.D. 527-565.

and ten pounds weight of gold * for depriving a neighbour of his view of the sea. Justinian enacted that these fines were to be handed over to the city's theatrical fund.

I do not, however, propose to trace our modern enactments right back to pre-Roman times. To-night we must confine ourselves strictly to our principal Act and its offspring, and I propose to curtail the subject still further by omitting to some extent such parts of the subject as have already been dealt with in the papers by Mr. S. Bylander on "Steel-frame Buildings in London" (February 13, 1913), Mr. Harold Cane "On the Stability of Brick Chimneys" (May 8, 1913), and Mr. W. Cyril Cocking on "Calculations and Details for Steel-frame Buildings" (February 26, 1914).

The Act should be clearly understood by all who read with average intelligence. It has to be administered by the London County Council and District surveyors. There are two important ways in which the Act needs amendment: (1) by making clear certain ambiguous or faulty wording, or wording that has been the subject of ambiguous or unsatisfactory decisions, and (2) by adding to or deducting from the existing provisions and requirements.

Let us remember the chief objects of the Acts are fourfold :—

(1) To secure a proper width and direction of streets.
(2) The sound construction of buildings.
(3) The diminution of danger arising from fire.
(4) The securing of more light, air, and space round buildings.

The principal Act is divided into sixteen parts :—

PART I is introductory, and its importance lies in Sec. 5, which contains 47 sub-sections or definitions, mostly re-enacted from older legislation. Several badly need amending and some new ones are required.

PART II deals with the formation and widening of streets, and is of secondary importance to this Institute of Structural Engineers, Architects, etc. Such societies,

* Present value about £600.

however, as the London Society and the Town Planning Society, with whom we are in the fullest sympathy, will do well to watch carefully any new legislation on this subject. Part II practically enacts that all new streets shall be at least the full clear width of 40 ft., open at both ends, and with a gradient not steeper than 1 in 20. It gives the Council power to require in certain cases an increased width up to 60 ft., but stipulates that in such cases compensation shall be paid for land or buildings or loss or injury due to the requirement of extra width of road above 40 ft.

It prohibits the erection of any new building, or extension of an old building, or any forecourt or enclosed space within the prescribed distance.* There is, however, a provision that old buildings within the prescribed distance * in old streets may be altered or re-erected on the same ground if a plan of the existing building is *first* submitted to the district surveyor and certified by him.

PART III enacts that no building or structure shall, without the consent of the Council, project beyond the general line of buildings. This line shall be defined, if required, by the superintending architect subject to appeal to the Tribunal of Appeal. Wide powers are given to the Council as to the conditions they may impose in giving consent to a projection beyond the general line. They may impose "any condition which the Council may deem it expedient to impose in the public interest." From their decision there is no appeal.

The City of London is exempt from this part of the Act.

PART IV, governing the naming and numbering of streets, has no particular interest for this Institute.

PART V requires open spaces to be provided to all *domestic buildings* except those used wholly or principally as offices and counting-houses.† It should be noted that a domestic building includes a dwelling-house, and *any other building* not being a public building or of the warehouse class. All habitable *basements* must have a proper open-air space of at least

* i.e. 20 ft. from centre of roadway in the case of a 40-ft. road, and 30 ft. in the case of a 60-ft. road.

† Compare with proposals in the Bill of 1905, p. 40 (Sec. 44).

100 sq. ft. Apart from this all domestic buildings are to have an open space at rear at least 10 ft. deep and 150 sq. ft. in area.

The height of the building in relation to the space at rear is regulated by an imaginary diagonal line inclining upwards at an angle of $63\frac{1}{2}^{\circ}$ from the rear boundary at the street level,* and Sec. 47 limits the height of all buildings to 80 ft. exclusive of two stories in the roof. It, however, permits a greater height to churches, and to other buildings if the Council's consent be first obtained.

We next come to PART VI of the Act, dealing with the construction of buildings, and consequently the most important Part so far as this Institute is concerned.

For the sake of clearness and brevity I propose to deal with this part and the schedules (at the end of the Act) together, and they may be conveniently epitomized as follows :—

Buildings shall be enclosed with walls of brick, stone, or other hard incombustible materials on concrete or other foundation,† properly bonded and solidly put together ‡ and of such thickness as the height § and length ‖ of the wall require to comply with Part I of the schedule (for domestic class of buildings) and Part II (for warehouse buildings). Footings are required to all walls, though the District surveyor is permitted to consent to their omission in certain cases where an external wall abuts upon another external or party wall (see Sec. 87).

Recesses and openings may be made in external walls to the extent of one-half the area of the wall above the ground story. There is no limit to the extent of the openings and recesses in external walls at the ground floor and basement levels.

Recesses may be made in a party wall to the extent of half its vertical face subject to, *inter alia*, the back of the recess being at least $13\frac{1}{2}$ in. in thickness.

* In the case of buildings erected in a street formed or laid out before 1894, the diagonal line may be drawn from a point 10 ft. above the street level.

† See First Schedule, Preliminary (1).

‡ Ibid., Preliminary (2).

§ Ibid., Preliminary (7).

‖ Ibid., Preliminary (8).

Woodwork must be kept back 4 in. at least from the external face of walls with the exceptions of door and window-frames and bressummers and story-posts.* Bressummers are to have at least a 4-in. bearing on piers or story-posts, in addition to the bearing upon any party or external wall. Woodwork must not be built into any party wall, and the ends of any wood beam or joist must be kept at least 4 in. from the centre of the party wall.

A parapet at least 1 ft. in height and 8½ in. in thickness must be provided to an external wall when a gutter formed of combustible material adjoins such wall.

Parapets are required to all party walls. In warehouses exceeding 30 ft. in height † the parapet must be at least 3 ft. in height and 13 in. in thickness. In all other cases 8½ in. in thickness and 15 in. in height is considered sufficient. The height of the parapet is to be measured from the roof, flat, or gutter of the highest building adjoining thereto, and above any lantern dormer or turret within 4 ft. of such parapet.‡

Chases (for pipes) may be made in party walls § if not more than 4½ in. deep and with a back at least 8½ in. in thickness and not less than 13 in. from an external wall, nor less than 7 ft. from another chase on the same side of the wall. Probably no section of the Act is more often contravened than this, and unfortunately the making good to the irregular chase does not result in the best work being ultimately obtained.

In a case recently under my observation a plumber had a chase cut 40 ft. high on the inside face of a wall immediately under a girder carrying 50 tons. Soon afterwards another man proceeded to cut a similar chase on the other side of the wall, leaving only 4 in. of brickwork, 40 ft. high, to carry, *inter alia*, 25 tons of girder-load.

The Act of 1894 contains no rules governing the *construction* of roofs,‖ but stipulates that they

* Secs. 55 and 56.

† Measured from the street level, see Sec. 5 (21).

‡ Sec. 59.

§ Sec. 60.

‖ Nor floors. Sec. 61 partly repealed and Sec. 62.

shall be covered with incombustible materials and shall not have a steeper slope than 47° for the warehouse class and 75° for any other class of buildings and that no domestic building shall contain more than two stories in the roof.

Chimneys of the ordinary domestic character account for no less than 22 subsections in the Act, mostly directed towards the prevention of the spread of fire. The only point I deem requiring your consideration is the support of the hearth.* The intention of the Legislature appears to be that it shall be supported on stone or iron bearers or other fire-resisting materials. In the fire at Sidney Street, Mile End, a fireman was fatally injured owing to the "stone hearth, laid on concrete, 6 in. thick," being supported wholly on light wood joists, which quickly burnt away and allowed the hearth to fall on the fireman's back.

This subsection requires verbal alteration, and is dealt with hereafter.

Chimney-shafts.—This subject is dealt with under Sec. 65, and, stated briefly, requires that the walls of chimney-shafts shall not be less than one brick thick for the topmost 20 ft., and shall be increased at least one-half brick for every additional 20 ft., measured downwards, with a taper of $2\frac{1}{2}$ in.† in 10 ft. with footings spread all round the base by regular offsets to a projection equal to the thickness of the enclosing brickwork at the base of the shaft and the space between the footings built in solid. The height of the shaft if square is not to exceed 10 diameters, or if round, or any other shape, 12 diameters.

A small digression may perhaps be permitted here in reference to the paper on the Stability of Chimney-shafts by Mr. Harold Cane read before this Institute in May 1913. Mr. Cane deplored the regulating of the construction of chimney-shafts in this cast-iron fashion, and gave three examples in which shafts if built as enacted would not have been satisfactory as regards stability. He further stated that in those towns in the north which had adopted by-laws regulating shafts the London Act of 1894 had been largely followed; but, said Mr. Cane, "*fortunately* in 60

* Sec. 64 (16).

† Equal to a batter on each side of $1\frac{1}{4}$ in.

per cent. of the towns concerning which observations were made no regulations were in force . . . . each individual case being judged on its merits." Now, I take the opposite view to that of Mr. Cane. The Act stipulates a minimum of its requirements, and in no way prohibits increased measures of safety. Granted that the three examples quoted are either of somewhat insufficient stability or slightly excessive weight, such disadvantages are far more than outweighed by the advantage of having definite rules for guidance. Remember we are dealing with London only, and the experience in London is that the stability of shafts erected in accordance with Sec. 65 is quite satisfacfactory, and I know of no failures. Prior to this Act (1894) the old Metropolitan Board of Works gave its sanction to each case.

Time forbids comment on the rules for habitable rooms,* construction of party arches and floors, though much difference of opinion exists as to the definition of "party."

Projections † such as cornices, porches, balconies, and architectural decorations must be of fire-resisting materials (with a few specified exemptions, the chief of which relates to shop fronts), and the extent of the various projections allowable are set forth.

Bay windows and oriels are permitted, with certain restrictions, in all streets of at least 40 ft. in width. In streets of less than 40 ft. in width bays and oriels can only be erected with the consent of the London County Council.

No water shall be allowed to fall from a building so as to drop upon or run over any public way.‡

Every building shall be separated § from the adjoining building by party or external walls, and, if over 1,000 sq. ft. in area and partly used for trade or manufacture and partly as a dwelling-house, the trade portion shall be separated from the dwelling-house portion by walls and floors constructed of fire-resisting materials, and all passages, staircases, and other means of approach to the part used as a dwelling-house shall be constructed throughout of fire-resisting materials. Provided that there may be constructed in the walls

* Sec. 70. † Sec. 73.
‡ Sec. 73 (7). § Sec. 74.

of such staircases and passages such doorways as are necessary for communicating between the different parts of the building, and there may be formed in any walls of such building openings fitted with fire-resisting doors.

This proviso badly needs verbal alteration, and is dealt with later.*

If the building exceeds 2,500 sq. ft. in area and contains separate sets of chambers, offices, or rooms, the floors and principal staircases shall be of fire-resisting materials.

Sections 75, 76, and 77 of the Principal Act were repealed by Sec. 16 of the London County Council (General Powers) Act, 1908, and Secs. 17 to 21 of the latter Act take their place. It will be more convenient to deal with these provisions here.

The size of warehouse buildings is restricted to 250,000 cub. ft., except where divided by proper division walls so that no one section exceeds the limit mentioned, and any openings in such division walls are to be fitted with iron doors.

Buildings shall not be united except with the consent of the Council unless they are wholly in one occupation and when considered as one building they conform to the requirements of the Principal Act.

"Wholly in one occupation" is a phrase liable to different interpretations, but it probably may be taken to mean, a householder may have lodgers, or a block of offices be let to different tenants, as there would still be only one legal occupier responsible for rates, taxes, inhabited house duty, etc.

Public buildings † are, "notwithstanding anything in this Act," to be constructed in such manner as may be approved by the district surveyor. This would seem to give unlimited power to the last-named, but in practice it may be taken that (so far as is generally applicable) the rules of the Acts must be followed and that the schedule for warehouse thickness of walls may be adopted,‡ unless the building is a steel-framed

* See page 31.

† See Sec. 5 (27).

‡ It is to be noted that in the regulations made by the London County Council under the Metropolis Management and Building Acts (Amendment) Act, 1878, with respect to theatres and other places of public resort, Regulation 5 requires that the thickness of walls shall be at least that prescribed by the London Building Act, 1894, for warehouse buildings.

structure, when, of course, the regulations of the London County Council (General Powers) Act, 1909, will be observed. This power of discretion to District surveyors is most useful to the designer and to the District surveyor. It allows little points of construction, such as so often occur, to be settled at a preliminary meeting or on the site.

It must, however, be remembered that this discretion only extends to *construction*. Lines of frontage, height, means of escape in case of fire, and all other points other than actual construction are still regulated by the various Acts. Sec. 80 lays down the rules as to the width and number of staircases, width of exit corridors, etc.

PART VII governs :—

(1) Permanent iron buildings or structures or any other building or structure to which the general provisions of the Act do not apply.

(2) Temporary buildings or structures, such as the Salvation Army Hall, erected for one month's use in Aldwych and Strand.

(3) Wooden structures (note the omission of the word "building"), including hoardings over 12 ft. in height, which may not be erected unless licensed by the Borough Council.

PART VIII deals with party structures and the rights of building and adjoining owners. A party wall may, *if both owners agree,** be built partly on the land of each, otherwise the building owner must place his wall wholly on his own land,† but in the latter case he has the right,‡ after one month's notice, to place the projecting footings and concrete on the land of the adjoining owner making compensation for such damage (if any) as he may cause to the adjoining owner.

Where an external wall is built against another wall, the district surveyor may allow the footings on that side to be omitted.

Subject to giving two months' § notice to the other

* See Sec. 87 (1), (2), and (3).

† See Sec. 87 (5). ‡ See Sec. 87 (6).

§ One month only in the case of a party fence wall.

side, either owner of a party structure may make good, underpin, raise, or pull down and rebuild any party wall or alter it so as to bring it into conformity with the Acts.* In the case of buildings having rooms or stories of such respective buildings intermixed, either owner may pull down such rooms or stories as are not in conformity with the Acts and rebuild in a proper manner. Again, he may pull down any party structure which is of insufficient strength for any intended new building and rebuild it of sufficient strength.†

These very drastic "rights" are, however, greatly modified (in practice, at any rate) by the proviso that in all cases of dispute the matters disputed shall be referred to a third surveyor,‡ and if his award is not accepted by both the building and adjoining owners an appeal therefrom may be made to a County Court,§ who may rescind or modify the award in such manner as it thinks just.

A small hospital had just been renovated when notice was served upon the governors by the building owner of the adjacent building that he intended at the expiration of two months to pull down the party structure which was (admittedly) of insufficient strength ‖ for the intended new lofty warehouse. The building owner was quite prepared to bear the whole of the expense and to temporarily screen off the hospital wards. The matter, however, went to arbitration, and the arbitrator decided that to take down the wall would cause unnecessary inconvenience ¶ to the governors, as, in his opinion, the old wall could be incorporated in the new. So the building owner's rights were of little value in this instance.

Where it is intended to erect a building within 10 ft. of a building belonging to an adjoining owner, and the foundation of such new building is carried down to a lower level than that of the adjoining owner, the latter *may* require the building owner to underpin and otherwise strengthen the foundations of his existing building.

In many cases this section (93) is overlooked alto-

* Sec. 88 (1) and (2).

† Sec. 88 (4) and (7).

‡ Sec. 91 (1) and (2).

§ In certain circumstances to the High Court.

‖ Sec. 88 (7).

¶ Sec. 90 (3).

gether. The building owner omits to give notice of his intention to build "within 10 ft.," contenting himself with the usual party wall notices. Sometimes the adjoining owner neglects to exercise his powers, being satisfied with the assurance of the builder that no harm will be done, and it is not until (perhaps a year later), fractures appear and possibly the London County Council serve him with a "Dangerous Structure Notice" that he realizes the importance of securing his rights.

It has been suggested that the District surveyor should have the same power as the adjoining owner in this matter. It would throw a considerable responsibility upon him, though, of course, it is one which he is well qualified to take. It certainly would safeguard the adjoining owner who neglects to look after his own interests by engaging a qualified architect or surveyor. But why should such neglectful persons be specially considered?

There have been several important cases in which the rights of owners of party walls have been involved.

It has been held that for the purpose of *ownership* a wall is only a party wall for so much of its height and length * as is used to separate adjoining buildings,† and that beyond the length or the height so used the wall may be (for the purpose of ownership) an external wall belonging wholly to one owner. For all other purposes, however, the wall for its whole height must still be regarded as a party wall, and must be carried up as a parapet above the highest building adjoining thereto,‡ and must contain no openings,§ though certain recesses || are allowable.

PART IX. Wherever it is made known to the London County Council that any structure is in a dangerous state, the Council require the District surveyor ¶ to survey it and report its condition, and to state what steps (if any) are in his opinion necessary for the safety of (1) the structure, (2) the public, and (3) the surrounding property.

* Sec. 58.

† With, of course, an additional 3 ft. or 15 in. as the case requires for the parapet.

‡ Sec. 59.

§ Except previously existing openings, which may be restored (see Sec. 101).

|| Sec. 54 (2).

¶ Or other competent surveyor.

The District surveyor is bound to make known to the Council any information he may receive as to a structure being in a dangerous state, but it does not appear to be his duty to search for such structures. Of course, if he happened to see any very dangerous structure it is obvious that he is morally (though not legally) responsible for any accident that may occur if he ignores it.

If the owner of the structure dispute the necessity of any of the requisitions of the Council, he may require that the matter be referred to arbitration. This is sometimes done, but more often the owner (if he neglects to comply with the notice to take down or repair) allows himself to be summoned before the magistrate, and the latter either makes an order for the work to be done forthwith or dismisses the summons.

If the District surveyor certifies a structure to be dangerous to its inmates, a Petty Sessional Court may direct constables to remove the inmates, who will be accommodated at the workhouse until they can make other arrangements.* One case in this City of Westminster occurred in 1908, where at one o'clock one day the District surveyor passing by observed an obvious recent extension of a bad fracture in the main front wall. Being unable to impress the resident owner with the urgency of the case, he hastened to Spring Gardens, telephoned for men and materials, and, with the Council's prompt and effective action, an order was obtained at Bow Street to remove the inmates. By two o'clock the constables had the people all out and blocked the street to pedestrians and vehicular traffic, and at three o'clock the first part of the building crashed *in*. It would have crashed *out* if the Council's men had not promptly averted such a calamity. The adjoining properties were in great jeopardy for some days, and eventually three were demolished. The trouble was chiefly due to a surreptitious alteration in the basement of the first house by the lessee, a " man of straw," who quietly walked away and left the various owners to suffer. He did, however, call at the District surveyor's office afterwards, but it was merely to inquire what compensation he

* Sec. 114.

was to receive from the Council for his demolished building !

The Act appears to require amendment so that such persons may be deterred from such callous disregard for the lives and property of others.

Fees are chargeable by the London County Council against "the owner" whenever a "Dangerous Structure Notice" is served, and would range from 18s. 6d. upwards to several pounds according to the area and number of stories of the building, the number of visits and reports as to progress made by the District surveyor, and the expenses the Council are put to if the case is taken into court.

PART X deals with dangerous and noxious businesses, and enacts chiefly that no person shall erect any building nearer than 50 ft. to a building used for any such business as is included in the list which follows :—

Manufacturers of matches or other substances liable to sudden explosion, inflammation, or ignition, or of turpentine, naphtha, varnish, tar, resin, or Brunswick black.

Blood or bone-boilers and any other like business which is offensive or noxious.

PART XI, broadly speaking, prohibits, except by permission of the Council, the erection of dwelling-houses on low-lying land * and so situate as not to admit of being drained by gravitation into an existing sewer.

An attempt has recently been made to include as low-lying land all land below the highest recorded high-water mark. This would be deplorable, as no finality would be possible. The present "highest" water mark might not be the highest, say, next year.

PART XII deals with sky signs, the removal of which were required within certain periods, and this part is, therefore, practically obsolete. By Sec. 5 of the London Government Act, 1899, the London County Council's powers under Sec. 134 of the London Building Act were transferred to the Borough Council. The powers are exercised in the City of London by the Corporation.

PART XIII opens by giving power to the London

* Below the level of Trinity high-water mark.

County Council to "appoint some fit person to be called the Superintending Architect of Metropolitan Buildings together with such number of clerks as they think fit."

Every building or structure and every work done to, in, or upon any building or structure (subject to the provisions of this Act) shall be subject to the supervision of the District surveyor.

The Royal Institute of British Architects may cause to be examined all candidates seeking certificates of competency to act as District surveyor, and no person shall fill such office unless he has received such certificate of competency from the said Institute or been examined and found competent in such other manner as the Council may direct.

The builder or other person causing or directing any work shall serve on the District surveyor a building notice stating the situation, area, number of stories and intended use of the building or structure, and the District surveyor shall survey the work and cause the provisions of the Act and by-laws to be duly observed, and he may enter and inspect such building, structure, or work at all reasonable times. If from the notice served on the District surveyor it appears any work contemplated will contravene the Act the District surveyor shall serve upon the builder or building owner a notice of objection, and the builder or building owner may appeal against such notice to a Petty Sessional Court.

If in the progress of the work anything is done in contravention of the Act, or anything is omitted which the Act requires, the District surveyor shall serve a "Notice of Irregularity" on the builder to amend the work within forty-eight hours.*

Then we come to the payment of fees,† upon which so much has been written and said.

The builder, or in his default ‡ the owner or occupier, is required to pay the fees laid down by the statute on (1) every new building, (2) every addition or

* The builder, however, merely has to amend the work, and not as in the Middle Ages to be "well blooded." I have heard of a small builder who considered he was being "bled" when he found the Act required him to pay the statutory fees, but the process certainly did not enfeeble him.

† Sec. 154 and Third Schedule.

‡ It has been held that the builder's "neglect or failure" is equivalent to default, and thereupon the owner or occupier becomes liable.

alteration. The amount is regulated by the area of the site covered by the building and by the number of stories, including, of course, the basements (if any). In the case of public buildings and tenement buildings these fees are increased by one-half. There are also included in the schedule eleven other fees chargeable to the builder, owner, or occupier in connection with various works.* Also there are the dangerous structure fees charged by the London County Council.

It is strongly felt by many that fees to District surveyors should be altogether abolished; that his duties are entirely for the public welfare, and he should be paid by salary; that it is iniquitous to empower the District surveyor to charge heavy fees for a small alteration on a large building, and several more or less powerful reasons are stated.

On the other hand, it is argued that the system answers very well; that with scarcely an exception District surveyors materially abate fees for small alterations on large buildings; that the building owner is the person who presumably profits by the work done, and that, therefore, it is fairer that he should pay the fees rather than the public, who have no interest in his building speculations.

Very much more than this has been said, but it is not, I think, a subject of special interest to this Institute. The London County Council may at any time † abolish the fees and pay salaries out of the County Fund, but probably it is deterred from so doing because (among other reasons) the amount required (some £50,000) would be a considerable tax on that fund.

PART XIV gives the London County Council power to make by-laws under twelve different headings.‡ That

* (1) Chimney shafts; (2) furnace and other flues; (3) cutting away chimney breasts; (4) certifying plans of old buildings; (5) attending at court; (6) stable construction under Sec. 70; (7) oriels; (8) trade stoves; (9) high-pressure pipes; (10) formation and closing of openings in party walls; (11) constructions under the public way.

† Sec. 158.

‡ (1) Regulation of plans; (2) forms of notice; (3) foundations and sites; (4) method and materials for foundations; (5) thickness, description, and quality of walls; (6) the dimensions of wooden bressummers; (7) dimensions of floor joints; (8) protection of ironwork; (9) woodwork in external walls; (10) plastering; (11) filling of excavations; (12) regulations for lamps and overhanging signs.

body has only availed itself of such power by making by-laws governing (1) foundations and sites of buildings; (2) the description and quality of the substances of walls; (3) the depositing of plans and sections; (4) quality of plastering; (5) mode of filling up excavations.

Other matters, as for instance the protection of ironwork from the action of fire, have been dealt with in later legislation, and for the sake of brevity will not be considered here.

PART XV enacts that all prosecutions (for which other provision is not made) shall be dealt with under the Summary Jurisdiction Acts, better known perhaps as Police Court procedure.

The Tribunal of Appeal * (first established under the London County Council (General Powers) Act, 1890) is reappointed, and consists of †—

One member appointed by the Secretary of State (usually a lawyer who is chairman of the Tribunal), now Mr. A. A. Hudson.
One member appointed by the Royal Institute of British Architects, now Mr. John Slater.
One member appointed by the Surveyors' Institution, now Mr. H. T. Steward.

Principally the matters adjudicated by the Tribunal of Appeal are—

(1) Appeals from the decision of the London County Council as to—

The formation and adoption of streets.
Open space in rear of corner buildings.
Prescribed distance.
Open space in rear of irregular sites.
Buildings beyond diagonal line.
Dwelling-house not abutting on street.
Old sites.
Cleared areas.
Greater height of buildings.
Conditions to Council's consent.

* Sec. 175.
† If the matter in dispute arises under the London County Council (General Powers) Act, 1909, an additional member is to be appointed by the Council of the Institution of Civil Engineers (see Sec. 25).

Means of escape from new or existing high or twenty-person buildings.
Construction of roofs of projecting shops.
Means of access to roofs.
Conversion of buildings in such manner as to contravene the 1905 Act.
Conditions affecting use of structural metal other than steel.
Construction of certain details in erection of steel-frame buildings.

(2) Appeals from the decision of the superintending architect as to—

General line of buildings.
In which street a building is situate.
Which is the front of a building.
Level of ground.

(3) Appeals from the decision of the District surveyor as to—

Frontage line.
Plan of old buildings—refusal to certify.
Construction of public buildings.

PART XVI is fitly headed Miscellaneous. It allows historically old buildings * to be sympathetically restored, subject to the consent of the Council being first obtained.

It recites a list of what constitutes an "offence † against this Act," with varying penalties, varying from 40s. to £20 and a daily penalty of the like amount.

Many buildings are partially exempt from the operation of the Act in varying degrees,‡ while a few set forth in Sec. 204 are wholly exempt, except as regards the building line. Gas companies are specially privileged.§

All exemptions and privileges, however, last only so long as the building is used for the purpose or retains the character by reason whereof it was so exempt or privileged.

No alteration may be made to any building if by

* Sec. 191. † Sec. 200.
‡ Secs. 201, 202, and 203. § Secs. 201 and 205.

reason of such alteration it will cease to be in conformity with the Acts, and no building, or part thereof, shall be converted or used for a different purpose than that for which it was originally constructed and legally used, unless when so converted and used for its new purpose it conforms with the provisions of the Acts relating to the class of buildings to which the building when so converted will belong.*

This brings us to the end of this epitome of the Principal Act of 1894; but before proceeding to the first Amending Act mention should be made of other enactments regulating buildings in the County of London, though they form no part of the London Building Acts, viz.:—

- Michael Angelo Taylor's Act.
- Highways Act, 1835.
- Theatres Act, 1843.
- City of London Sewers Acts, 1848 and 1851.
- Common Lodging-houses Acts, 1851 and 1853.
- Metropolis Management Act, 1855.
- Metropolis Management (Amendment) Act, 1862.
- Valuation (Metropolis) Act, 1869.
- Metropolis Management and Building Acts (Amendment) Act, 1878.
- Metropolis Management (Amendment) Act, 1879.
- London County Council (General Powers) Act, 1890.
- Metropolis Management Amendment Act, 1890.
- Public Health (London) Act, 1891.
- London County Council (General Powers) Act, 1897.
- City of London (Sewers) Act, 1897.
- London Government Act, 1899.
- London County Council (General Powers) Act, 1900.
- City of London (Various Powers) Act, 1900.
- Factory and Workshops Acts, 1901 and 1907.
- City of London (Public Health) Act, 1902.
- Housing and Town Planning Act, 1909.
- Cinematograph Act, 1909.
- London County Council (General Powers) Act, 1910.

* Sec. 211.

The London Building Act Amendment Act, 1898, gives the London County Council power to enforce the setting back of any new portion of a building erected within the prescribed distance from the centre of the roadway. Section 14 of the Principal Act, which was intended to have this effect, was upset by the Court of Queen's Bench in the case of "London County Council *v*. Aylesbury Dairy Company" (1898), hence the new legislation, and that section is now repealed. The remainder of this Amendment Act deals chiefly with the service of notices, etc.

The London Building Acts (*Amendment*) *Act*, 1905, in its "Bill" * form was a drastic and sweep-

* The main points in this Bill not touched upon later were as follows :—

(1) The increase of width of new streets for carriage traffic from 40 to 50 ft., and making the Act apply to the City of London as elsewhere (so that if the Corporation wanted to form a new street the London County Council's consent would have to be obtained), and a general revision of the rules governing the formation of new streets.

(2) An amendment of Sec. 13 (5), by which a person obtaining a certified plan of an old building can only, by virtue of such certificate, build to the same height as before unless he gives up so much of his land to the public as is within 20 ft. of the centre of the road. So far this is excellent, but further suggested amendments would have practically meant the forfeiture by owners of all land (and buildings if and when demolished) within the prescribed distance of 20 ft. from the centre of roadway.

(3) Prohibiting the erection of buildings or a part thereof beyond the general line of building.

(4) Prohibiting the erection of any building or structure within 35 ft. of the centre of the roadway.

(5) If the Council think it in the public interest it may define a building line (in any street new or old apparently) not more than 75 ft. from the centre of the road, and thereafter no building or addition may be formed within such defined line.

(6) The exemption of office buildings in the main Act (Sec. 39) was to be altered from building used "principally as offices or counting houses" to used to "a greater extent than three-fourths of the cubical extent thereof." Any basement of such building was to have open spaces not less than 5 ft. wide, and 100 sq. ft. in area on every side of such basement containing a window. The minimum depth of open spaces about buildings would be increased from 10 to 15 ft.

(7) Basement windows of any habitable room were to have an unobstructed light angle of 63½ degrees (see Clause 47). Inspection chambers were to be in the open air and not inside a building (see Clause 50).

(8) No building other than domestic could have been built to a greater height than 16 ft.) within 12 ft. of the open space provided in connection with any domestic buildiug.

(9) No dwelling-house could be raised if within 20 ft. of the centre of the roadway unless such raised portion were set back the 20 ft.

ing measure; but, meeting with considerable opposition, only a small, though important, portion passed into law. We must not stay to consider the "might have been" if the Bill had passed *in toto*, though it will be referred to in some of the few amendments submitted later for the consideration of this Institute.

This Amendment Act deals entirely with the protection against fire, and enacts shortly that—

(1) Every new building which is either (*a*) more than 50 ft. in height, measured from the street level to the topmost *floor* level; or (*b*) a building in which accommodation is provided for more than twenty persons, shall be provided, to the Council's satisfaction,* with such means of escape in case of fire as can be reasonably required.†

(2) In every existing building exceeding 50 ft. in height (measured as before) in which sleeping accommodation is provided for more than twenty persons or which is *occupied* by more than twenty persons the Council * *may* at any time require the building to be provided with proper means of escape. It will be seen that this is (and rightly) far less stringent than in the case of new buildings.

The case of smaller and less lofty buildings is next dealt with.

Shops projecting more than 7 ft. beyond the main building now have to be roofed with fire-resisting materials not less than 5 in. in thickness, and lantern lights and ventilating cowls are only permitted under certain restrictions. The object is obviously to mitigate the evil of the flames from a shop on fire sweeping up the front of the building and preventing the escape of the inmates by the windows on to ladders.

Houses and dwellings ‡ not exceeding 30 ft. in height; dwelling-houses occupied by not more than two families; and existing buildings not having more than two stories above the ground story, are not

(10) Openings in walls were not to exceed in width three-fourths of the length of the wall *in each story*. This would have had a drastic effect on shop windows, the exemption of basements and ground stories having been removed.

(11) Habitable rooms were to have a minimum height of 9 ft.

* Subject to appeal to Tribunal. † Sec. 7.

‡ Factories and workshops are dealt with under another Act—the Factory and Workshops Act, 1901.

required to make any provision, but *all* other existing buildings are now required to provide means of access to the roof (in a position approved by the District surveyor) by means of a dormer, or an automatic trap-door, or other approved access, which must be always readily available. Guard-rails on the roof are required where reasonably practicable and necessary to prevent persons slipping off the roof.

"The owner" is bound to do the work, but he may apply to the County Court * to apportion the cost he has incurred "among all the several persons entitled to any estate or interest in the building . . . regard being had to the terms of any lease." I have only once heard of advantage being taken of this power.

The exempted buildings largely follow those of the Principal Act, but a building three-fourths † of which is used for a bank or insurance office is (subject to certain provisions) exempt from the provisions of this Act of 1905. Common lodging-houses are also exempt.

Attached to the Act is a schedule of fire-resisting materials, which, however, may only be considered as such when used for certain specified purposes.

The Steel-frame Act, or, to give it its proper title, Part IV of the London County Council (General Powers) Act, 1909, was devised to meet the growing demand for the use of steel construction. Hitherto the Legislature had given no recognition of steel framing as applied to buildings as a whole, though, of course, steel stanchions and girders (included in the Principal Act with story-posts and bressummers) were regulated when (but only when) carrying a wall or walls. Internal stanchions and girders (and for that matter internal brick piers) were not within the supervision of District surveyors.

The new Act now makes it lawful to erect buildings wherein the loads and stresses are transmitted through each story to the foundations by *a skeleton framework of metal*.‡

Though this framework, where external, must be

* Sec. 20. † Of its cubical contents. ‡ Sec. 22.

protected with 4 in.* of fire-resisting material, it allows a great gain of space in the lower stories. For instance, a warehouse building of the full statutory height would require (if the walls were of considerable length) brickwork 3 ft. thick in the basement, whereas now with steel construction 14 in. is all that is required.

All rolled steel used in the construction of steel skeleton buildings shall comply with the British Standard Specification. The steel framing shall be capable of independently sustaining all the loads, including wind pressure,† except such as are taken by the party walls. The thirty-five subsections of Sec. 22 deal fairly exhaustively with the details of construction of the steel framing and include rules governing the protection of the steelwork from fire, the depth of girders in relation to span, and security against buckling; the number, spacing, etc., of rivets and bolts; thickness of plates and bars in pillars and the jointing thereof; the provision of proper bases thereto and the filling in of hollow pillars with concrete or closing the ends with riveted metal plates.

All structural steelwork is to be cleaned of all scale and loose rust, and painted both before and after erection, but proper cement-wash may be substituted.

Then follow rules governing the calculation of dead loads and superimposed loads with a useful table of the latter, which may be summarized as follows :—

Dwelling-houses	70 lbs. per sq. ft.
Offices	100 ,, ,,
Workshops and retail shops ...	112 ,, ,,
Warehouses	224 ,, ,,
Roofs with greater slope than 20°	28 ,, ,, (to include wind pressure)
Roofs with slope of 20° or less...	56 lbs. per sq. ft.

Subsection 19 of Sec. 22 allows a reduction in the calculated superimposed load in buildings of more than

* Sec. 22 (3). † Sec. 22 (20).

‡ Provided that if the load on floor or roof exceeds that given in the table provision shall be made to carry such load in compliance with Sec. 22 (22).

two stories in height, which, I think, is clearly exemplified in the following :—

	Superimposed Load per Square Foot calculated for Beams.	Superimposed Load per Square Foot to be provided for by Pillars, Foundations, and Piers.
Roof	56	56
Twelfth floor ...	100	100
Eleventh floor ...	100	95
Tenth floor	100	90
Ninth floor	100	85
Eighth floor	100	80
Seventh floor ...	100	75
Sixth floor	100	70
Fifth floor	100	65
Fourth floor	100	60
Third floor	100	55
Second floor	100	50
First floor	100	50
Ground floor	100	50

This reduction is not allowed in the case of buildings of the warehouse class, in which provision must be made for the full superimposed loads in each story.

Tables are given for calculating the working stress on (*a*) cast-iron pillars and (*b*) mild steel pillars and giving the maximum working stresses (other than those due to wind pressure) for various ratios of length to least radius of gyration when the ends are considered as fixed, or one end fixed and one hinged, or both ends hinged.

The loads on the natural ground are not to exceed—

1 ton per square foot if on soft clay, or wet or loose sand.
2 tons per square foot if on ordinary clay or confined sand.
4 tons per square foot if on compact gravel, London blue clay or chalk.

I believe this is the first time any such arbitrary classification of natural soils has been attempted. To a certain extent it is useful, but foundations vary so

NOTE.—The author is unable to say what is the intention of the Legislature in enacting Sec. 22.

greatly, especially in central London and near the river, that much discretion and experience is needed to classify the soil and decide the load per square foot it may safely be expected to carry. In all important or heavy structures trial holes should be made and the thickness of the bed upon which the structure is built should be tested. In an important building now being erected I was asked to approve a "good hard gravel bottom," but on testing it with a crowbar it proved to be 6 in. in thickness and to rest on soft clay.

The pressure on concrete foundations * shall not exceed 12 tons per square foot. Many of my hearers will agree that this is quite a safe allowance. In the case of excellent concrete foundations amply reinforced with steel, a far greater load might well have been allowed subject possibly to the District surveyor's approval. As it stands the section necessitates a greater spread in the base plate of a stanchion than is convenient unless a steel grillage is used. Practically it discourages the use of reinforced concrete foundations and encourages the use of steel grillages.

Brick pillars are recognized, and their height restricted to twelve times the least width of such pillars, if properly supported laterally with a minimum of 13½ in. and a maximum safe load of—

12 tons per square foot if of blue brick in cement mortar.
8 tons per square foot if of hard brick, including London stock.
5 tons per square foot if of ordinary brickwork.†

The Legislature appears to have lost sight of the main purpose of this new Act, viz. to permit and encourage the construction of steel-framed buildings in which the loads and stresses are transmitted *by a skeleton framework of* METAL to the foundations. If brick pillars are to be allowed *ad lib.*‡ the building

* Sec. 25.

† It is a little difficult to know what is meant by "ordinary brickwork," as it obviously is not intended to include the ordinary hard bricks in use, including London stocks. Surely the Legislature did not intend it to mean soft bricks. From their use in brick pillars carrying steel-frame buildings may we be delivered.

‡ Why not brick walls?

ceases to be framed in steel. It is not clear when a steel-framed building ceases to be such within the meaning of the Act.

I am told by a colleague of a case where the architect came to the District surveyor with designs for a steel-frame building. Upon its being pointed out to him that the steelwork was far from being of the strength required under the new Act, he calmly said he would make his external walls of the full thickness required under the Principal Act of 1894 and thus obviate the necessity for complying with the Steel Frame Act of 1909. Had he intended to design and build his walls to carry his steelwork, it would have been bad enough,* but he sought to further overload his already weak construction by adding the extra thickness of brickwork. The Act requires to be made quite clear in this respect.

A new phase of the 1909 Act, and a very necessary one, is the requirement that *a copy of the calculations of the loads and stresses* shall accompany the notice to the District surveyor, in addition to such plans and sections of sufficient detail as he may reasonably require.†

The London County Council has power to modify or waive certain but not all requirements of the new Act, and should it refuse to do so the applicant may appeal to the Tribunal. I have not heard of any such appeal, and it is curious that the conditions which appear to me most needing latitude are not included in those the London County Council may waive.‡

Section 23 gives the London County Council power to make regulations with respect to the construction of buildings of reinforced concrete. The section does not in itself give the right (as more than one lecturer

* The bond of the brickwork would have been hopelessly at fault by the intersection of the steel framing.

† On one occasion after laboriously checking and completing some 1,200 sheets of drawings and calculations (about one month's work), and requiring certain corrections, the consulting engineer (who, by the by, has read an admirable paper on steel framing before this Institute), calmly writes that the design is *somewhat* altered, and he enclosed 316 sheets of amended calculations ! !

‡ Requirement that sufficient rivets must be provided in machine-planed bases to take up entire stress.

in another place has asserted) to build in reinforced concrete, and it is only within the last few months that such regulations have been practically approved.

It must be remembered that this Amendment Act of 1909 is a permissive Act.* It permits the owner, if he chooses, to substitute steel framing in lieu of stone or brick external walls of the statutory thickness. In all other respects whatsoever he is still bound to comply with the requirements of the Principal Act and the various Amendment Acts.†

SUGGESTED AMENDMENTS.

There are a score or more of minor amendments obviously to be desired in the Principal Act alone, and a few in the respective Amendment Acts, but obviously this is not the time or place to deal with them in detail, e.g. if I were to begin—

Section 65 (1) delete the word "taper" and substitute "batter"

I should expect to have the hall to myself in five minutes.

What I think we can do with greater utility is to consider a few amendments on broad lines without attempting to put the suggestions into the exact phraseology necessary for new legislation, and, after having discussed and decided what we want, to leave the whole matter until the proposed new Act is within the range of practical politics. Then, when that time comes, we shall be ready to take our place with the older institutes in aiding Parliament and the London County Council to put such an Act on record that the next generation will secure proper width and direction of streets, sound and economic construction, reasonable safety from fire, and sufficient light and space about buildings.

The Bill of 1905 aimed at all these, but failed chiefly, I venture to suggest, because it was too drastic, and would have laid a very heavy burden on the owners of to-day while leaving the entire advantage to be reaped by a future generation.

* A disputed point. † Sec. 27.

I would first suggest that the definition of a new building should include any addition (or alteration) to an existing building when such addition equals in cubical extent the cubical contents of the existing building incorporated with the new. I know a case where a new church was built but the old vestry was left. The builder described the work as an addition to the vestry. Of course such an extreme case is merely ludicrous, but it is sometimes difficult to know where to draw the line. Hence my suggestion.*

The next is more important, and has often been discussed under Sec. 13 (5).

If a certified plan of an old building in a narrow street (i.e. under 40 ft. in width) is first obtained, it is now permissible to rebuild (or raise) that building to a height of 80 ft. plus two stories in the roof.

This means that in a street 13 ft. in width, with cottages on each side, it is lawful to erect buildings in lieu thereof as shown (Fig. 1).

This produces a most depressing and insanitary street, and should not be allowed. I suggest that the old building may be re-erected to the same height as before, but that any increase in height be prohibited except to such portions of the building as are at a greater distance than 20 ft. from the centre of the road (see Fig. 2).

It would be better if the plans of *all* old buildings immediately abutting upon any street (of whatever width) were certified before demolition.

Section 59 I suggest amending to read: All party walls and all external walls within 5 ft. of any other building to be carried up to form parapets at least $8\frac{1}{2}$ in. in thickness and at least 15 in. in height in every part above the roof, flat or gutter, and in the case of warehouse buildings exceeding 30 ft. in height such thickness to be increased to 13 in. and such height to be increased to 3 ft.

Section 74 (2) requires the portion of a building used as a dwelling-house to be separated, *inter alia*, by walls from the part used for trade. But there may be necessary doorways in such walls, and there may be

* This suggestion has already been incorporated in the 1905 Act, Sec. 6 (1) (v), but is only applicable to that Act. It should be applicable throughout the suggested new Act

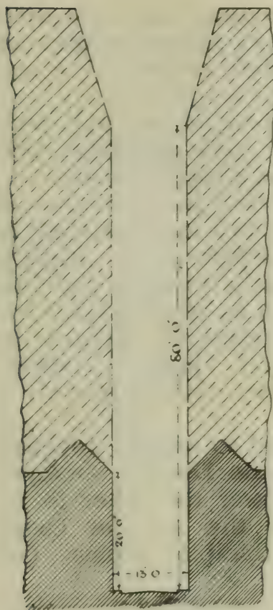


FIG. 1.

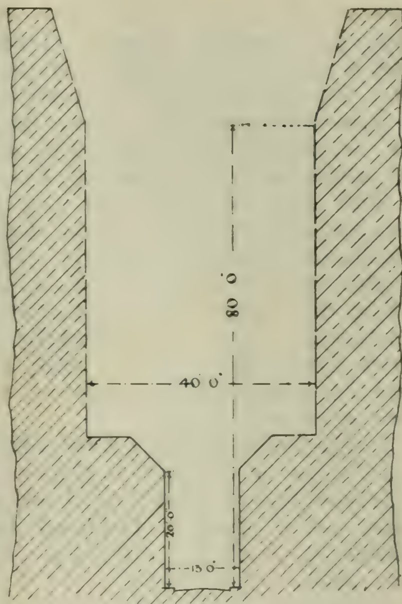


FIG 2.

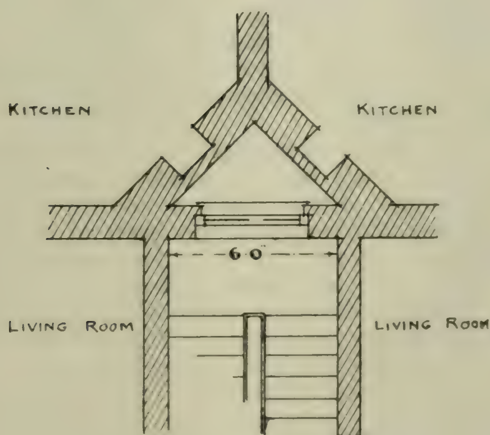


FIG. 3.

formed in any walls of such building openings fitted with fire-resisting doors. This is rather ambiguous, and is capable of being read in different ways. I suggest that it be altered to read :—

Provided that there may be formed in such walls any necessary doorways, and all such doorways shall be fitted with self-closing fire-resisting doors.

The Act requires that the principal staircases of buildings occupied by more than two families shall be ventilated upon every story. The plan given (Fig. 3) is that of the staircase of a fairly modern building producing about £480 per annum as small flats. The ventilation (!) is provided by means of a triangular open space of 9 sq. ft. and 40 ft. high. I always find the windows shut tight to keep out the bad air in the so-called open space.

I suggest that the Act be altered to require adequate ventilation to the satisfaction of the District surveyor, provided that nothing shall empower the latter to require more than is required by the Act for courts within a building lighting habitable rooms (see Sec. 45).

Section 64 (10) might well be extended to include the words, "No flue shall be within 2 in. of the centre line of a party wall."

Arising out of the fatal fire at Sidney Street, Mile End, I suggest amending Sec. 64 (16) to enact that the hearth or slab shall be supported upon iron or other fire-resisting materials, wholly independently of any wood bearers or supports, to the satisfaction of the District surveyor.

Section 70 of the Principal Act requires that every habitable room shall have a window opening directly into the external air, or into a conservatory. Presumably, if the window is in the front of the building, it lights into the street, and if at the rear into the open space at rear with a minimum depth of 10 ft. But windows in flank, external walls often have the walls of adjoining properties within 3 ft., and much higher than the top of the window * (see Fig. 4).

I suggest that every habitable room shall have one

* In one case a habitable room has a window within 1 ft. of another external wall, and the latter is at least 20 ft. higher than the top of the window.

or more windows opening directly into the external air *with an unobstructed angle of light of* $63\frac{1}{2}°$ from the sill level (see Fig. 5), calculated as if the window were one-tenth of the floor area.

It will be noted that I have deleted the alternative of lighting and ventilating into a conservatory. If the alternative is to be retained at all, it should be

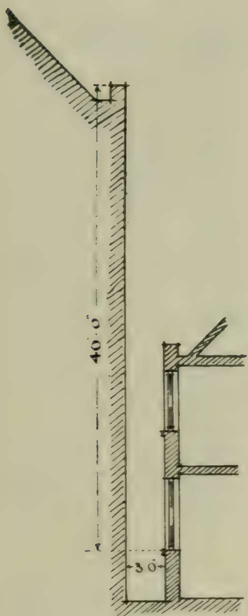


Fig. 4.

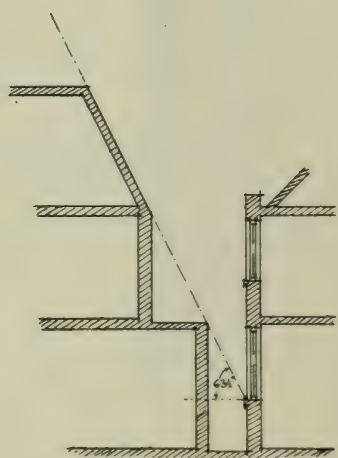


Fig. 5.

with the proviso that the conservatory must be maintained as such and used for no other purpose. Plans were brought to me showing a yard at rear to be covered in with a glass roof to form a pot-house of certain flourishing licensed premises. On my objecting on the ground that the only window of a private kitchen opened directly into the proposed addition, and would cease to be properly lighted and ventilated, the builder took the plan away to reconsider the matter. Next morning he brought it back quite triumphantly with the word " pot-house " crossed out and

" conservatory " substituted. Evidently he had acquired a *little knowledge* of the Building Act.

There is scarcely a more prolific source of the spread of fire from one building to another than the common lighting court (see Fig. 6).

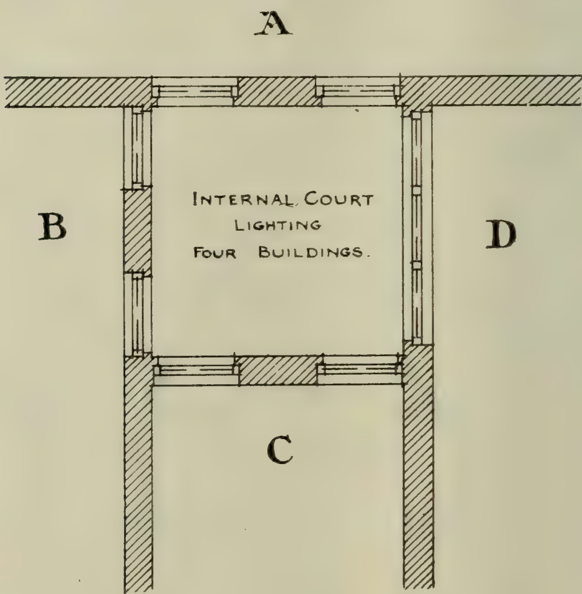


FIG. 6.

" A " caught fire, but the flames were extinguished in twenty minutes, but the fire went through " B's " window.

" B " was burnt out and spread the fire to " C," fortunately with little damage to " C."

" D " escaped.

I suggest that any windows or other openings opening into an internal court in common with one or more other buildings shall be constructed and glazed with fire-resisting materials and that not more than one-third of the area of any such window may be made to open.* Any skylight or lantern light in

* In every habitable room it is required that a portion of the window (equal to one-twentieth of the floor area) shall be made to open. At first sight my suggestion would appear to contravene this, but it will not be found to be necessarily so in practice.

any such court shall be constructed of fire-resisting materials and glazing, and only the vertical sides of such lantern lights may be made to open.

The London County Council in the Bill of 1905 suggested in Sec. 84 (2): No opening . . . shall be made in any external wall of any building in such a manner that an imaginary line, 30 ft. in length, drawn from any point in the plane of any existing opening in any external wall of another building and in the direction of such first-mentioned opening and inclined to the aforesaid plane at any angle not exceeding 135° would touch or intersect the plane of the first-mentioned opening at any point therein.

The meaning of this you may find difficult to grasp on first reading, and I have therefore prepared a sketch in illustration (see Fig. 7). The shaded area of the wall is the portion in which no window or other opening may be made by reason of the existence of the window "A" in the adjoining building.

I must not linger over other amendments of the Principal Act and the Amendment Acts of 1898 and 1905, but devote my remaining minutes to suggested improvements of the Steel Skeleton Amendment Act of 1909 and the Regulations on Ferro-Concrete.

Section 21 of the 1909 Act defines the expression "pillar" (unless otherwise stated) as "a metal pillar, and shall include all columns and stanchions or an assemblage of columns or stanchions properly riveted or bolted together."

In connection with pillars there are, I submit, one or two very desirable amendments. For instance, in the case of a stanchion consisting of a rolled steel joist with plates there is no regulation as to what extent the plates may project beyond the flange of the joist. It would apparently be no contravention of the Act to use a 10 in. × 6 in. R.S.J., with one or more 16-in. plates riveted thereto. The breadth of the plates would increase the least radius of gyration of the pillar over the least radius of the R.S.J. alone, and consequently a greater stress per square inch could under the Act be placed on the pillar, though obviously if the plates project 5 in. beyond the flange they would (unless they were of great thickness) be incapable of carrying a heavy stress.

It is desirable to limit the projection of plates beyond the flange, and possibly some member will be ready to suggest a practical formula to give effect to this.

In this connection, too, I may draw attention to

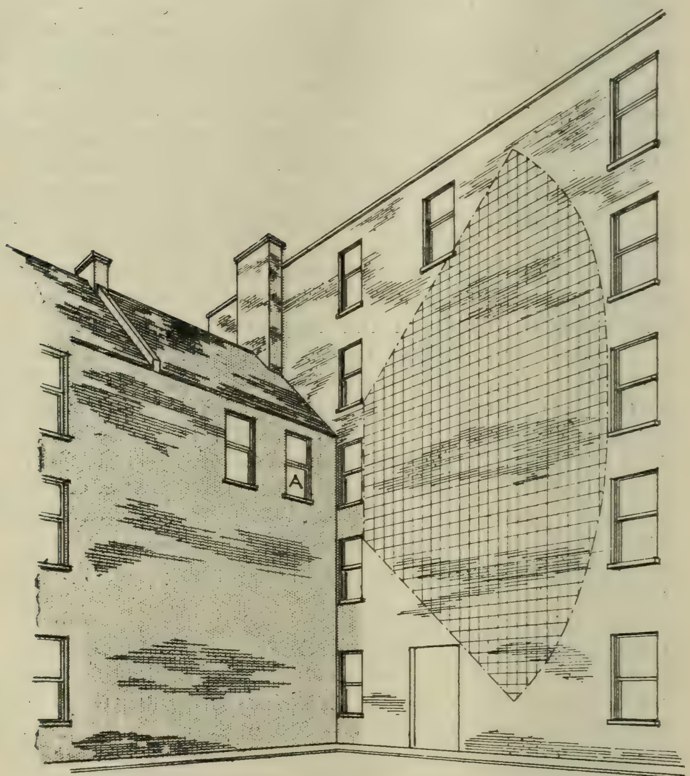


FIG. 7.

the fact that it is desirable where there is any considerable overlap of plates to use "tack" rivets if there are two or more thin plates. But I know of nothing in the Act to require these, unless, indeed, one could consider that the definition quoted could be applied, and one could insist that it was not a

metal pillar because it was not *properly* riveted together.

Perhaps some member will be good enough to define "properly."

Section 12 (*c*) is somewhat ambiguous, but it is generally understood to mean that there shall be sufficient rivets at the base to transmit the whole of the load from the stanchion to the base-plate.

This apparently disregards the bearing of the pillar directly on the plate, and I submit that if a pillar is riveted together and then the base is machine-planed before the base-plate is put on there is no necessity for such a number of rivets, and that if (say) sufficient rivets were put in to take 60 per cent. of the load ample security would be obtained.

In the case of a solid steel column no rivets at all are (or can be) used.

Another point worth consideration in connection with pillars is the jointing or splicing. The regulation as to this is contained in sub-section 12 (*b*). Here, again, we find the word "properly," which occurs no less than three times. No rule is given as to the total sectional area of rivets to be used; if the ends of the pillars are machine-planed at right angles to the axis, an excellent joint is obtained, but there is nothing in the Act to require machine-planing. Experience shows that in pillars with sawn ends it is seldom that a joint can be close-butted over its whole area, and it is therefore desirable that the cover-plates and rivets should be sufficient to carry a large proportion of the load from the upper length to the lower.

Very little discretion is given to the District surveyor under this Act, and no indication is given as to whose interpretation of "proper" or "properly" is to be the decisive one, notwithstanding that the words occur ten times within the first 14 sub-sections.

It is to be noted that under Sec. 34 power is given to the Council to modify or waive any of the requirements of this sub-section 12 (*b*) as to joints in pillars.

Sub-section 7 limits the span of a girder to twenty-four times its depth, unless the calculated deflection is less than $\frac{1}{400}$th of the span.

That some such regulation is necessary there can

be no doubt. In plans which have been submitted to me (not under this Act) I have seen $4\frac{3}{4}$ in. × $1\frac{3}{4}$ in. R.S.J.'s, 3 ft. 6 in. apart over a span of 18 ft. 6 in. But I think it would have been better to make the maximum span in all cases dependent upon the calculated deflection. Under the present regulation one may use a 10 in. × 6 in. R.S.J. to carry a distributed load of $10\frac{1}{2}$ tons over 20 ft. span, when the deflection would be $\frac{1}{320}$th, whereas if one had to carry only $8\frac{1}{2}$ tons over a span of 20 ft. 6 in. it would be necessary to use a 12 ft. × 6 in. R.S.J. in order to limit the deflection to $\frac{1}{400}$th.

Another point on which considerable difference of opinion has been expressed is Clause 10, which requires that rivets shall be used in all cases where reasonably practicable. This has been sometimes held to mean that rivets must be used wherever it is possible for the men to handle the tools; but I think the word "practicable" has a wider meaning than this. For instance, if a connection has to be made through a thickness of 4 or 5 in. I do not consider hand-riveting to be practicable, as the rivet would be considerably cooled by being put through the hole before it was possible to begin riveting and sufficient power could not be used in hand-riveting on the site to make the rivet fill the hole and to put a proper head on it. In such a case I should consider bolting to be more "practicable."

Then there is a question which, like the poor, is always with us—viz. that of end-fixing of stanchions. When is an end fixed? It is very desirable that there should be uniformity of practice in this respect, so that the contractor in estimating can know what he must provide for.

Section (25) limits the pressure on concrete foundations to 12 tons per square foot. This is equivalent to about 186 lbs. per square inch. which to reinforced concrete engineers will sound an extremely low figure. In the regulations for reinforced concrete a stress of 600 lbs. per square inch is allowed, and it is not very apparent why such a low limit should be fixed in this Act. Some alteration in this figure is desirable, though, of course, it would be useless to increase it to the figure allowed for reinforced concrete. The

stress allowable on the ground to some extent limits the stress on the concrete, because, unless the concrete was very thick, the load per square foot on the soil

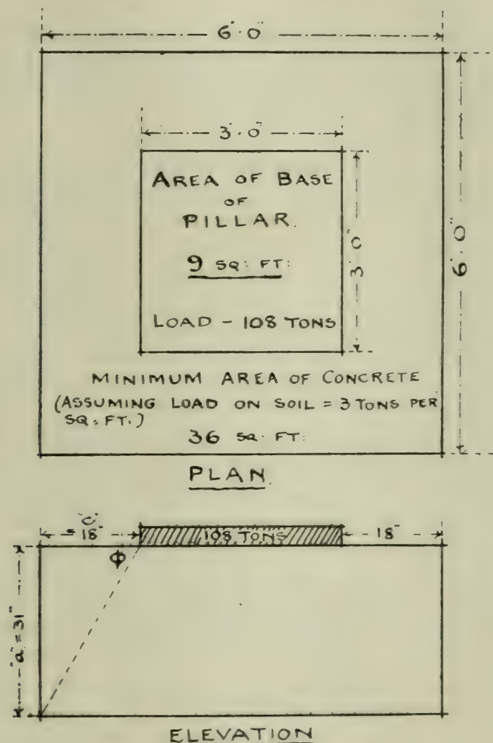


FIG. 8.

Minimum depth of concrete required to reduce the load on the ground to 3 tons per square foot = "a."

Projection of concrete beyond steel base = "c."

"c" = 18 in.

ϕ = angle of dispersion of load through concrete (say 60° from the horizontal).

Then "a" = "c" tan ϕ,

$= 18 \times 1{\cdot}7321$,

$= 31$ in.

could not be reduced to the limit allowed by the Act if the stress on the concrete were greatly in excess of 12 tons per square foot (see Fig. 8). To put

a great thickness of concrete is not economical, because it is merely adding further weight on the ground without any equivalent gain in strength.

Reinforced Concrete Regulations.

The Regulations of the London County Council with regard to reinforced concrete are divided into ten parts, as follows :—

Part I (*General*) describes the meaning of the term "reinforced concrete," specifies the application of the regulations and the nature of the materials to be used generally in reinforced concrete buildings, prohibits the use of any reinforcing metal for conducting electrical currents, and provides that copies of the plans and calculations, together with particulars of the materials to be used, are to be deposited with the District surveyor at the time when notice is served upon him under Part XIII of the 1894 Act.

Part II consists of data to be used for the purposes of the Regulations. It contains a list of superimposed loads to be allowed for on floors to be used for various purposes and roofs of different slopes. It requires provision to be made for wind pressure, and specifies the maximum allowable ratio of span to the depth of a beam or cantilever. It prescribes the bending moments to be allowed for in beams and slabs with various degrees of end fixing under different dispositions of loading. It specifies the maximum stresses to be allowed on concrete of various proportions, and in steel. It makes provision for anchoring the ends of all tensile and shear reinforcement, and specifies the modular ratio to be assumed in beams and pillars.

Part III (*Beams and Slabs*) contains regulations as to the minimum diameter and disposition of the steel reinforcement and the minimum thickness of slabs, and regulations as to notation and formulæ to be used in calculating the resistance moment in beams and slabs.

Part IV relates to pillars and struts, and defines the terms "pillar" and "strut" and contains regulations as to the minimum and maximum diameter of reinforcing rods and their disposition and minimum

area, regulations as to notation and formulæ and tables to be used in calculating the permissible load with various degrees of end fixing.

PART V contains regulations as to walls, specifies the minimum thicknesses allowable, and the maximum area of openings permitted. It is to be noted that party walls and division walls are allowed to be constructed of reinforced concrete, and need not be of the thickness prescribed by the 1894 Act.

This is a variation from the provisions of the 1909 Act in the case of steel-framed buildings, which require all party walls to be of the thickness prescribed by the 1894 Act.

All brickwork, stonework, and plain concrete are required to be executed in Portland cement and mortar, and the allowable pressures on brickwork so built are specified. No regulations are given as to the allowable pressure on stonework built in cement mortar.

PART VI contains regulations as to the allowable pressure on various beds of natural ground, and also limits the pressure on plain concrete in foundations.

These are similar to the provisions of Sec. 22 (24) and (25) of the 1909 Act in the case of steel-framed buildings.

PART VII (*Protection*) contains regulations as to the minimum cover of concrete over the metal reinforcement.

PART VIII (*Materials and Testing*). This part contains regulations as to the quality, size, and proportions of the materials to be used for making concrete, with a table of the ultimate compressive stresses which must be attained for various proportions at the end of one month and four months after mixing. Regulations are also made as to the manner of mixing and placing the concrete. Nos. 165 to 167 give directions as to the quality and treatment of the metal reinforcement.

No. 168 is printed amongst the regulations as to "steel" though it would apparently come better under the next set of regulations headed "Tests and Testing," which specify the maximum test loads to be applied; such tests are not to be made within ninety days of the date of laying the concrete.

PART IX contains regulations as to "Formwork or Centering."

PART X relates principally to workmanship, providing that the work shall be carried on as continuously as possible, that it shall be protected from too rapid drying and from frost, and that concrete shall not be laid when the temperature is less than 7° above freezing.

No. 185 is a most essential regulation prohibiting the cutting away of concrete for pipes or any other purpose in such a manner as would reduce the strength of any part of the structure below the standard set up by the Regulations.

The remaining three regulations deal with woodwork, etc., fixed in or on the concrete.

There does not appear to be the same difficulty in determining whether the Regulations as to reinforced concrete will apply to a particular building as there is in determining whether a building is or is not a steel-framed building. Regulation 2 states that "These regulations shall apply *only* to the construction of buildings of reinforced concrete in which the loads and stresses are transmitted through each story to the foundations by a skeleton framework of reinforced concrete, or partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls."

Section 22 (29) of the 1909 Act provides that "It shall be lawful to make any addition to or alteration of or to do any other work to in or upon a building in accordance with the provisions of that section provided that the loads and stresses in the part of the building so added or altered, etc. . . . are transmitted from the roof to the foundations by a skeleton framework of metal . . . and the provisions of this section shall in all respects apply to such part of a building as if the same were a separate building."

There appears to be no similar provision in the regulations, though from the wording of Clause 6 it appears to be contemplated that additions or alterations may be carried out under the Regulations. In my opinion it would have been well to have inserted a similar clause to Sec. 22 (29). It would also have been better to make the Regulations applicable in all cases where reinforced concrete is used. In the present

circumstances there would appear to be no regulation as to reinforced concrete floors in ordinary or steel-frame buildings.

In the Regulations as originally drafted there was a provision similar to that in Sec. 22 (18) of the 1909 Act, viz. "In every building of the warehouse class a notice shall be permanently exhibited in a conspicuous place on each story of such building stating the maximum superimposed load per square foot which may be carried on any part of the floor of such story." This provision does not appear in the latest revision of the regulations. In my opinion such a regulation is even more desirable in the case of reinforced concrete than in the case of steel framing, because in the former it is impossible to ascertain by inspection the strength of a floor.

No allowance is made for high-tension steel, so that apparently those systems which use mesh and other high-tension steel reinforcement would be inadmissible in buildings erected under these regulations. In the R.I.B.A. Second Report such steel is recognized, with a provision that the stress on it shall not exceed one-half the stress at the yield-point of the steel, and a maximum of 20,000 lbs. per square inch.

It is also desirable to make a provision both under the Regulations as to reinforced concrete and in the Act as to steel-frame buildings that the drawings and calculations should be deposited with the District surveyor some time (at least one month) *before* the work is to be commenced. In the present circumstances, as notice is only necessary two days before the work is commenced, sufficient time is not given to the District surveyor to go through the scheme and check the loads which will come on the foundations before the foreman is clamouring at the door wanting the District surveyor to pass the bottoms for the stanchions and pillars.

The regulations as to testing do not specify who is to require tests of the completed structure. Testing may be found necessary "by reason of any sign of weakness or faulty construction appearing." But the contractors' idea of "signs of faulty construction" may be very different from the District surveyor's. Who then is to decide? The New York regulations provide that—

"The contractor may be required to make load tests on any portion of a reinforced concrete structure within a reasonable time after execution." It is also noticeable that the New York regulations require the construction "to sustain safely a load of *twice* the superimposed load for which it was designed," whereas the London County Council Regulations limit the test load to $1\frac{1}{2}$ times the superimposed load for which the construction was designed.

The London County Council regulations as to protection are considerably less stringent than the New York regulations. The latter require a minimum cover of 2 in. of concrete in columns and girders, $1\frac{1}{2}$ in. in walls and beams, and 1 in. in floor slabs; the former only require $1\frac{1}{2}$ in. in pillars, 1 in. in beams, and $\frac{1}{2}$ in. in slabs.

The allowable stresses in concrete of 1 : 2 : 4 mixture under London County Council Regulations and New York regulations are compared below :—

	L.C.C.	New York.
	Pounds per square inch.	
Extreme fibre stress on concrete in compression	600	650
Concrete in direct compression	600	500
Shearing stress	60	40
Adhesion stress when bars hooked at ends ...	100	80
Adhesion stress, bars otherwise effectively anchored	60	80

When I was asked to give a short paper on "the whole subject of the London Building Acts, including the Steel-frame Act and the Reinforced Concrete Regulations," I felt I was asked to do an impossibility. My paper is very far from dealing exhaustively with the subject and I am fully alive to its shortcomings. I can only, however, submit it for what it is worth, and trust that the discussion to follow will bring out many useful suggestions for future amendments.

In conclusion, I would again emphasize the position that I submit this Institute should adopt. It has already been recognized as an authority by the Legislature in the Amendment Act of 1909, and it will, I trust, be ready to render able assistance when a new Act or another Amendment Act is in contemplation.

DISCUSSION

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., etc.) :—Building by-laws form a perennial source of discussion. There are perhaps no regulations more difficult to make and none that give so little satisfaction to those concerned in obeying them. From time to time the objectors fall into line, and modifications ensue. The time now appears to be ripe for another general consolidation of the various Acts and Regulations, and this is a good opportunity to ventilate grievances. As to the details in the paper, with regard to the footings required for brick walls under the London Building Act, it is worthy of note that in the north of England it is customary to build without footings, although in the London Building Act we are told they are essential. The District surveyor's fees on repairs often press with undue hardship, and sometimes exceed the cost of the repairs themselves. I have had to pay fees upon the measurement of a whole house for merely resetting a small arch over a doorway. No doubt many others have had similar experiences. As to the load on foundations, where clay is concerned, the pressure is of much less consequence than provision against shifting, and nothing short of a reinforced concrete raft can be considered satisfactory if the bottom of the foundation is within 10 ft. of ground-level. The author, towards the end, asks for suggestions as to the greatest projection of flange plates in a built-up stanchion. In my own practice I make eight times the thickness the limit of projection, and the same rule applies to girders. Where there are two or more plates I should consider "properly" riveted meant "tack" rivets with not more than 6-in. pitch. I agree with the author as to the riveting in the base plates, but it is seldom that the base is machine-planed, before it is connected to the base plate, and that is why the whole stress must be capable of being taken by the rivets. With regard to riveting through a thickness of 4 or 5 in., I should like to remind the author, that it is not so many years ago, that there was no machine riveting, and good work was quite possible by hand through this thickness. All that is necessary is, that

the point should be cooled, before the rivet is driven, so as to cause it to swell in the centre under the blows of the riveters. I agree with the author as to the desirability of a notice being permanently exhibited on every floor of a reinforced concrete warehouse, stating the maximum superimposed load for which the floor was designed, and the Council of the Concrete Institute pressed for this regulation to be included. I believe the answer to it was, that the London County Council had no power to make such a regulation.

Mr. H. D. SEARLES-WOOD, F.R.I.B.A. (Vice-President C.I.), proposed a vote of thanks to Mr. Osborn Hills for his epitome of the London Building Acts, and also for his valuable suggestions. There are one or two points in his epitome with which, he said, I did not quite agree. For instance, he refers to the third surveyor in the party-wall section as an arbitrator ; but he is not an arbitrator ; they are three equal surveyors, and any two can make the award. In his list of Acts which regulate building in London I notice a curious omission, and that is the Chimney Sweeps' Act, which regulates the size of a flue. Fourteen inches by nine inches is laid down in the statute, and anybody who detects a builder making a flue of any other dimension than that can get a fine inflicted and have half of it as a common informer.

Mr. HILLS :—Is that in force in London ?

Mr. SEARLES-WOOD :—Certainly.

Mr. HILLS :—Thank you. I think I can see my way to earn a little money over this !

Mr. SEARLES-WOOD :—Yes, the Act was passed by the good Lord Shaftesbury, in order to prevent the chimney-sweepers' boys being sent up to sweep chimneys.

Mr. W. G. PERKINS :—I believe the dimension is 14×14.

Mr. SEARLES-WOOD :—I think it is 14×9, but it may be 14×14. In regard to Mr. Hills' historical

opening, some years ago I read a paper on the influence of building by-laws on design, and I referred to those celebrated Babylonian laws. They were the old *Lex Talionis,* an eye for an eye, a tooth for a tooth. If the owner's mother-in-law was killed by a house falling down, the architect's mother-in-law had to be killed too ! To show the influence of by-laws upon design, that was the period when the Pyramids were designed, and you can quite understand the family influence brought to bear on the architect to get things quite safe. With regard to the suggestions, I hope we are not going to have another Amending Act. I think the time is ripe now for considering the thing on an absolutely new basis and bringing the whole matter into accord. One of the great things we should do in a new Building Act would be to avoid that terrible delay in getting the approval of the London County Council. Things are now crystallized to such an extent, that it should not be necessary to spend all the time, that we *do* spend at the present moment in getting our plans passed. If there were reasonable regulations laid down, the District surveyors ought to be able to see the work as it goes along without these terrible delays, from which we all suffer so much at present. With regard to the wording of the Acts, surely it would be possible now for us to have a little more scientific treatment of the subject and not so many schedules. Let it be for the designer to shew, that he has properly calculated his structure, and then give him a free hand and do without all these schedules. I was talking over these things with a friend of mine the other day, and we were looking at that old book in the City of London Record Office, which contains FitzAlwin's Assize of building regulations, which were collected in the reign of Richard II, and were old even then, and there we think is the very earliest schedule, certainly in England. It is laid down there that every party wall shall be 3 ft. thick. I suppose that was in the time when they did not have mortar. There is another curious thing laid down in it, and that is that if a man wanted to build a party wall, and the adjoining owner did not want to build the wall, if the adjoining owner gave 3 ft. of his land the

building owner could build the party wall on that 3 ft. of land and would not be able to recover any of the cost from the adjoining owner. That seems to be a reasonable sort of thing.

Mr. HENRY LOVEGROVE, F.S.I., A.R.I.B.A., District Surveyor :—I quite agree with my old friend Mr. Searles-Wood, that we should have a new Building Act, and not a mere amendment. It would be very much better for everybody, if we had a new Act. People come to me time after time and say, "What does this mean?" and I have to explain it to them, because, you know, District surveyors are constantly at it and know these things. It would be very much better to have a new Act, and certain things in the present Act might be omitted, including that rigmarole about skysigns, which, to my own very great regret, have been abolished. I used to make £22 a year out of them! There are certain clauses in the Act, which require to be absolutely amended, because they are so badly worded. That clause noticed by Mr. Hills about parapets could be very much better worded and the whole thing made clearer than it is. For myself I should like to see the question decided as to when a party wall ceases to be a party wall, for at present the matter seems to me to be rather absurd. I know of a case of a warehouse building, 50 ft. in height, and it was built in such a way that there was a low shed, 12 ft. in height, on the other side belonging to the adjoining owner; and it is absurd to make a man carry up a wall 3 ft. above the roof of the taller building. Another very absurd thing is where you have two water-closets built and are obliged to put a parapet between them, because the only chance of a water-closet getting on fire is when the gentleman, using it, smokes. The thing is so impressed on the ordinary bricklayer, that I have come across cases where he puts a parapet on the front as well. In a small building like that it seems absurd to enforce the same regulations, as you would enforce for a warehouse.

Mr. W. G. PERKINS, District Surveyor (Member of Council C.I.) :—First of all I should like to second the vote of thanks.

The present Acts undoubtedly require to be amended and codified, in order that they should, first of all, be more easily understood by persons, who are building in London, and, secondly, so as to admit of more recent, improved, and scientific methods of building being adopted freely, without the necessity of going to the London County Council for special sanction in individual cases as must now be done, under section 82 of the London Building Act, 1894, if it is desired to build in a manner not prescribed by that Act. In amending the Acts the language and real meaning intended should be made so absolutely clear, that no doubt should arise. I had almost said that, at the present time in nearly every other line of the various clauses a double meaning can be inferred. Let me give an instance. Look at section 64 (18) in Part VI of the Act. It says there "a flue shall not be built in or against any party structure, unless it be surrounded with new brickwork at least 4 in. in thickness properly bonded." It was necessary to go to the High Court in order to find out what that means. A builder erected a flue against an existing party wall. That party wall was of quite recent construction and built with good brickwork, but the part owner of that party wall said to the builder, "You are not putting 4 in. of new brickwork all round that flue." They went to court over it, and Mr. Justice North said the words "new brickwork" meant brickwork which was new at the time of the building of the flue. It does not matter that you are building the flue against a wall of new brickwork; you must put another 4 in. of brickwork at the back of the flue. It is most ridiculous. Some of you gentlemen will no doubt argue, that the District surveyor should have more discretion in the various cases, that arise under the Act, but are you sure that that is a good thing? A would use his discretion in the way you would like and would meet you half-way and say: "That's all right. The construction is good; I will allow it"; but B, the next District surveyor, might say: "I don't like that; I want twice as much as what you propose." If you give him discretion in these matters, you may be very hardly hit. Discretion is a two-edged sword. I think you want to base your rules

upon common sense, and then in the language of the Act make it so clear what you mean, that the builder will know exactly what to do. Definitions should be made clearer in a great many of the cases. Let us look at the definitions of domestic buildings and of warehouses. The expression "dwelling-house" means a building used wholly or principally for human habitation. The expression "domestic building" includes a dwelling-house "and any other building not being a public building of the warehouse class." If you look up a building of the warehouse class, it is defined as being, *inter alia*, a building which is neither a public building nor a domestic building, so that you work in a circle. A domestic building is any other building which is not a warehouse, and a warehouse is any other building not a domestic building! That wants amending. Although many people may not know it, there is an exceedingly good definition of a building given in a leading case which is seldom referred to, that of Stevens *v*. Gourley, and if I may, I will read the decision of Mr. Justice Byles. He says: "What is the meaning of the word 'building'? It often happens that the verb 'to build' is used in a wider sense than the substantive 'building,' as to build a carriage, or to build a ship, and it is said birds build nests; but none of these things when built would be called a building. It is a well-established rule that words must be construed according to their ordinary meaning; and though it is difficult—I may say impossible—to define the word 'building,' yet it is easy to say whether this or that thing is or is not a building. What, then, is the ordinary meaning of the word 'building'? Without pretending to define it, I think it is usually understood to be some structure or erection of considerable size, intended to be permanent, or at least to last for some time, whether let into the ground or not. A church built of iron or of wood, a house, or a stable, or a coach-house, is evidently a building; but, on the other hand, a birdcage with a handle for lifting it off the ground, or a wig-box, is not a building." I do not think we want a definition of a building after that, because I should be quite satisfied with it, if I had to take a case into court. In Part III of the

1894 Act I would ask, why should the City be exempt from the law in regard to the general line of buildings? In the City you ignore section 22 altogether. If there is a general line of building, there is nothing in the City to prevent you building in advance of it. In regard to Part V, why should huge modern office buildings, erected to accommodate hundreds of clerks, be exempted from the provisions of the Act, which require proper open spaces about domestic buildings? The ordinary small dwelling-house has to have a certain amount of space about it, and I think the same rule should apply to the large buildings, where so many people are employed. The dark, badly ventilated rooms, with which one meets in those offices, call for a most drastic alteration in the law, and the provisions of section 45, relating to courts within buildings, should also be made to apply. Public buildings when of the nature of domestic buildings, such as large hotels, are doubtless subject to sections 41 and 45, and I have always treated them accordingly. In the case of the London County Council *v.* the Rowton Houses, the judges laid it down in a dictum, that there was no reason why a public building should not at the same time be a domestic building. As to the meaning of the words "wholly in one occupation," I am sorry I cannot agree with Mr. Hills. I am referring now to section 18 of the General Powers Act of 1908, which prohibits the uniting of buildings unless wholly in one occupation. Mr. Hills says, he thinks a block of offices let out to separate tenants may be considered to be a building in one occupation. That is not so, because the tenants if they have an agreement for more than twelve months become "owners." They only cease to be "occupiers" when they are "lodgers," and as a rule the person holding an office in a building is not a "lodger." As to the interpretation of section 78 of the Act of 1894, there, again, I cannot quite agree with Mr. Hills, that the District surveyor has unlimited discretion. I discussed this point rather fully some time ago in this room, and, briefly, my opinion is this, that the District surveyor must see that the whole of the rules of the Act are complied with in the case of a public building, but he has

power to ask for anything further that he thinks necessary in the way of construction. If you do not adopt that view, what interpretation are you to place upon section 68? That section enacts, that floors of the lobbies, corridors, passages, etc., must be of fire-resisting construction. Section 80 is another rule of construction. If therefore section 78 means that the District surveyor can do as he likes, he can set aside section 68 and section 80. I submit that he cannot. With regard to the rights of building and adjoining owners, the third surveyor (the umpire that Mr. Hills referred to), surely misdirected himself as to the meaning of the Act. The Act gives the building owner certain rights, and I do not think the umpire or any surveyor can take away from the building owner those rights. The case of Leadbitter *v.* the Marylebone Borough Council deals with the point in this way. An award was made by the surveyors, by which it was awarded, that the defendants should have *the right* at any time, without notice, to raise the party wall as they might desire. It was held in the Court of Appeal, that the award, so far as it purported to give the defendants *the right* to raise the party wall, was made without jurisdiction and was of no effect, and that defendants could not interfere with the party wall without giving notice as building owners under the provisions of the London Building Act of 1894. I think by analogy our third surveyor had no power to take away *the right* given to the adjoining owner to pull down that defective wall and rebuild it. It is admitted, that the wall was defective. On the question of fees, I heartily agree with Mr. Hills, when he suggests, that we should be paid by a salary.

MR. OSBORN HILLS:—I do not think I suggested that.

MR. PERKINS:—I was rather under the impression that you did. I certainly think we should be paid by salary, in order to avoid the nuisance of having to collect fees. It really is a nuisance. Professor Adams says he has been charged a large fee on a small work. It is perhaps rather hard on Professor Adams, but when you come to average fees all round they are quite reasonable. From experience I find,

that the average fee varies from thirty shillings to two pounds per work. Sometimes a small work on a large building takes months. And I would ask, is it fair to request the surveyor to take a correspondingly small fee, when by the slowness of your operations he has had to keep the premises under observation an unreasonable length of time? In dealing with open spaces about buildings, it is at present permissible in a street laid out before 1894 to erect a dwelling-house, so as to cover your entire site to a height of 16 ft. and then provide your open space above that level. That can result in back-to-back houses, a most undesirable state of affairs, that should surely be repealed in any amending Act. There is another little point that I would like to see put right. We often get slates slipping off a roof into the street, sometimes injuring people, and I think it would be well to enforce the construction of a parapet to the front or flank of any building, where it abuts on a public way or comes within a certain distance of the public way. I have had that happen in my district.

Mr. BYLANDER :—I have had some, too.

Mr. PERKINS :—With regard to Mr. Hills' plumber, I think it should be made unlawful to cut away brickwork, without a certificate from the District surveyor. It is a common practice. I have found it done by the best contractors. They will cut away footings and concrete all along a wall, in order to lay a drain close to it, and on several occasions I have found brickwork cut away, in order to pass pipes through a main pier. I would like to draw attention to the way, in which exempted buildings are increasing in number. Mr. Davidge drew attention to that in his paper before the Royal Institute, and showed how from a few buildings a number of years ago the exemptions have crept up to—I forget the exact figure, but it was quite a large number. I think everybody should be subject to the law, and if such were the case we should not have police buildings, railway-station buildings, and others jutting out in front of the building line in all our main thoroughfares. In the Bow Road you can see it; in the Clapham Road and in the Brixton Road you see these

buildings brought out to the public way many feet in advance of the adjoining dwelling-houses. It takes away the "view of the sea!" On page 88, towards the bottom, there is a little error, I think. Mr. Hills says: "Internal stanchions and girders (and for that matter internal brick piers) are not within the supervision of District surveyors." If your internal girder is a bressummer, it is subject to section 56. The question of the end-fixing of stanchions was first brought up by Mr. Cocking, and it was suggested that there should be a meeting of the various societies to try and come to some agreement as to what constitutes a fixed end. I wish Mr. Hills, who is now the secretary of the District Surveyors' Association, would write a letter to Mr. Etchells and try and make some arrangements for a first meeting. On page 103 Mr. Hills gives a diagram in regard to pressure on concrete foundations. By a little calculation I find from the load given that we get a maximum bending moment of 91,000 in.-lbs. on that concrete. By a rule given by Sir Benjamin Baker the concrete shown by the diagram would have a resistance moment of 192,000 in.-lbs., so that that thickness is well on the safe side. I can bear out, what Professor Adams says in regard to clay foundations. When I was District surveyor in Hackney, I had some underpinning done close by Stamford Hill, where there is a clay foundation. The District surveyor had no jurisdiction as to depth for underpinning so long as the soil is solid, and in one instance such an underpinning was done, and the excavation was 3 ft. from the surface of the soil. The next year it had to be done again, because the earth under the concrete had shrunk and caused cracks to appear again in the building. They then took the underpinning to a depth of 10 ft., and no further trouble was experienced. I disagree, that no more legislation should be passed with respect to skysigns, because if we do not have some legislation, skysigns will be revived.

MR. E. FIANDER ETCHELLS, Assoc.M.Inst.C.E., A.M.I.Mech.E., F.Phys.Soc. (Member of Council C.I.) :—Those builders, architects, and engineers

from the provinces who so frequently, having work in London, find their way to headquarters and ask what the Building Act requires of them might with profit read Mr. Hills' paper. It may take many years before any suggestions made to-night find themselves part of the law of the country, but the earlier they are made, the greater the chance of a full and complete consideration. Professor Adams has spoken of the requirements of the London Building Act in regard to footings. The London County Council has power to consent to the omission of footings, and the District surveyors under certain circumstances have power to consent to the omission of concrete under the footings. In regard to the suggestion by Mr. Searles-Wood, that there should be no more schedules, I think that is too happy a dream to be realized. In the Assize of FitzAlwin, apparently the single requirement of a 3-ft. thickness for party walls of whatever height was sufficient, but since that date much water has flowed under the bridges, and year by year has seen increasing complexity of building construction. Our legislation must follow some of the complexity of the actual buildings. Schedules must be still adopted. But that still leaves open the question as to where those schedules shall be found. Shall they be in the Act of Parliament itself (and Acts of Parliament are rather difficult to alter once enacted), or shall they be schedules submitted to the technical institutions, and capable of easier revision if necessary?

SIXTIETH ORDINARY GENERAL MEETING

THURSDAY, APRIL 8, 1915

THE SIXTIETH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, April 8, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

THE PRESIDENT :—I very much regret to have to announce the death of one of our honorary members, M. Edmond Coignet, one of the great pioneers of reinforced concrete, and it is particularly sad following so soon after that of M. Considère. A vote of condolence from the Council will be passed to the family, and a biography will probably appear in the TRANSACTIONS.

The business of this evening is to complete the discussion on Mr. Hills' paper, entitled "The London Building Acts, with some suggested Amendments," and I will ask Mr. Woodward kindly to reopen the discussion.

DISCUSSION

MR. WILLIAM WOODWARD, F.R.I.B.A., F.S.I.: —I remember perfectly well—and I dare say a good many of you do—the preliminaries of the Act of 1894, and at that time there were those in authority at the London County Council who desired to legislate at the expense of the owners of property. They un-

doubtedly *did* frame some of the sections of the Bill in a confiscatory manner. Well, the Bill was amended from time to time by discussions at the two Institutions, the Surveyors' and the Royal Institute, and at last we got the Building Act of 1894; and the Building Act of 1894 may, and sometimes does, interfere with the value of the ownership of property; but I must say, speaking personally, that on the whole I think the various sections of the Act protect the public from some of those great drawbacks and nuisances, which were in existence before its passing. I remember that the late Mr. H. H. Collins, who was a District surveyor, did an immense amount of good with regard to the present Act. He stuck to it religiously, and to Mr. H. H. Collins we owe a great deal for the smoothness of some of the sections of the Act. Now Mr. Hills hopes for a new Act, and so do I, and so do most of us, and I trust that when that new Act is passed it will contain in itself, without reference to other Acts, by-laws, regulations, etc., all that is necessary for carrying on the work of building in London. It is a great bother. As soon as you begin, as you think, to grasp this particular Building Act of 1894, you are referred to this Act and to the other Act, until it becomes a labour to know precisely what the Building Act of 1894 means. On page 3 of his paper Mr. Hills has a reference to Zeno, who lived A.D. 500, and the Building Act of the Emperor governed the elevations and distances of contiguous private buildings in Constantinople. I suppose we may gather from that, that at that time they had some idea of the "law of rights of light." It does not say anything about the Prescription Act in A.D. 500, but there is no doubt, that at that time they had some idea of light as affected by buildings. When I read that, and Mr. Hills took me back so far, I had an idea, that I could perhaps make a few quotations of an earlier date, and I looked through the Apocrypha. I do not suppose many of you gentlemen ever *do* look through the Apocrypha, so I will read a quotation or two. In the First Book of Esdras, chapter vi., verses 9 and 10, we have this: "Building an house unto the Lord, great and new,

of hewn and costly stones and the timber already laid upon the walls. And those works are done with great speed, and the work goeth on prosperously in their hands, and with all glory and diligence is it made." In the Second Book of Maccabees, chapter ii., verses 29, 30, and 31, you will find these words: "For as the master builder of a new house must care for the whole building: but he that undertaketh to set it out and paint it must seek out fit things for the adornment thereof, even so I think it is with us. To stand upon every point and go over the things at large, and to be curious in particulars, belongeth to the first author of the story, but to use brevity and avoid much labouring of the work is to be granted to him that will make an abridgement."

It appears to me that the verses of the Apocrypha have anticipated what Mr. Hills proposes. He is going to make an "abridgement." Talking of "abridgements," I have from time to time pitched into the Local Government Board. I took the trouble to count the words in the Local Government Board Model By-laws in their print of 1912, and I counted 21,578 words. I then went through the whole of the same by-laws and altered them in red ink, and I have told the Local Government Board, that I can get all that is practically needed in their by-laws in 2,198 words! So there is a difference between 21,578 words in Whitehall and 2,198 words by the individual, who is now addressing you. Mr. Hills makes a very valuable epitome of the Act of 1894, and to make an epitome of that Act must have entailed upon him, considering the shortness of his paper, a very great deal of labour, and to those, who have not the time, to go through that Act word by word, the epitome will be found of very great service. On page 81 Mr. Hills says: "If in the progress of the work anything is done in contravention of the Act, or anything is omitted which the Act requires, the District surveyor shall at once give 'notice of irregularity' on the builder to amend the work within forty-eight hours." I remember, before the Act of 1894, the difficulty was, that the building owner was put to very great cost in going on with the work, and the District surveyor was not able to

stop it, until an absolute contravention occurred. I do not know, even now, whether he is able to do this, but Mr. Hills will perhaps tell us. I am not quite sure whether the District surveyor can stop a building owner contravening the Act, unless he has really contravened it. Suppose, for example, a man is proposing to build a wall of insufficient thickness. I will take it now, that the District surveyor cannot stop that work, until he sees the wall, as actually building, is of insufficient thickness. The right course is, to go to the District surveyor with your drawings, before you start your work, and he can then say, " No, that is in contravention of the section," and so you are put right, before involving your clients in the cost which you would otherwise entail.

MR. W. G. PERKINS, District Surveyor for Holborn :—The power of giving notice of objection should be extended, to meet the case you mention. If it appears from the builder's notice, that the Act is going to be contravened, the Act says, the District surveyor *shall* serve a notice of objection, but the builder never *does* say he is going to contravene the Act.

MR. WOODWARD :—On page 78 I shall join issue with Mr. Hills on a disputed point. It is in reference to party walls and external walls. I have had two or three tussles with District surveyors about this particular section. Mr. Hills says, there have been several important cases, in which the rights of owners of party walls have been involved, and in the last five or six lines of the central paragraph on that page he says : " For all other purposes, however, the wall for its whole height must still be regarded as a party wall and must be carried up as a parapet above the highest building adjoining thereto and must contain no openings, though certain recesses are allowable." I have always regarded a party wall as a party wall to the height—giving an allowance for the 15 in. or 3 ft. of the parapet—only where it separates buildings, and above that height I consider that it is an external wall, and I have put window openings in the upper part of that wall ; and until I am pulled

up by Mr. Hills or some other District surveyor I shall continue to put windows in the upper part of such a wall as I refer to. Mr. Hills will perhaps have something to say about that, but that is the only section where I venture to differ from him as to its construction. I believe there is a great deal of difference of opinion amongst District surveyors themselves as to that particular question. On page 82 Mr. Hills refers to the question of District surveyors' fees. I am on the Borough Council of Hampstead, and on several occasions we have had communications sent to us complaining of the District surveyors' fees. If I remember rightly, some of the complaining parties came from Southwark. I have always defended District surveyors, because, first, the fees are statutory and the District surveyors are not responsible for them, and next, although in some cases they may get fees which may be considered large, in other cases they get fees which are much too low for the services rendered. I hope that with these absurdly low fees, coupled with the fees which are a little more profitable, the District surveyors are satisfied. The question has also arisen as to whether the District surveyors shall be paid by salary. Of course, that is a matter for themselves. In some districts the emoluments are sufficient, but in others they are very unremunerative, and perhaps when the new Act comes into force this question of whether District surveyors shall be paid by fees or by salary will be raised. On page 90 I must quote a few lines : "Tables are given for calculating the working stresses on (*a*) cast-iron pillars, and (*b*) mild steel pillars, and giving the maximum working stresses (other than those due to wind pressure) for various ratios of length to least radius of gyration when the ends are considered as fixed, or one end fixed and one hinged, or both ends hinged." I could no more work that calculation out than I could fly in the air, and now we know what the District surveyors have to do, to master this new work, you will agree with me, that they deserve a great deal more remuneration than they get ! I have only carried out one or two buildings on the steel-frame structure lines, and all I have been able to tell my clients, when they have

asked, what benefit they have got from this form of construction, is that "instead of having a 14-in. wall we have done it with a 9-in. wall and so saved the superficial area." But apart from that I am not quite sure whether I should not stick to my old-fashioned way of building and leave the steel-frame structures to be dealt with by the younger generation of architects. In regard to reinforced concrete, that is a matter which I am sorry to say I do not understand. I have never carried out a building in reinforced concrete, but I take it that one of the most important parts of this reinforced concrete business must be the careful mixing of the concrete. I can quite understand, that any deviation from this careful mixing must be a very important factor in reinforced concrete, even assuming and admitting that all the calculations are perfectly accurate. The most skilful clerks of works and foremen ought to be employed, to see, that the work is properly done. In regard to the new Act, I am quite sure the District surveyors can render most useful service in its draughting, and I trust that, practically, the total elimination of the "lawyer" will take place. The by-laws of the Local Government Board to which I have referred were framed by lawyers, and that accounts for the 20,000 words in lieu of the 2,200; and if you get lawyers on this new Building Act you will find all the same divergence of opinion as to the meaning of the sections, all the same openings for litigation and trouble, which one finds in present Acts of Parliament. Another little thing, which is of some importance, is the question of "punctuation." The lawyers will tell you, you must not punctuate an agreement, because if you do another interpretation may be placed upon the wording. You want to avoid another interpretation, and let us take, for example, the Workmen's Compensation Act. If the draughtsmen of that Act and of other Acts had for their sole object future and continuous litigation, all I can say is, that they have been singularly successful. I trust, that in a new Act, the draughting will be done by District surveyors, with the help of the architects and of the two or three Institutions, which have so much to do with the work.

Mr. EDWARD DRU DRURY, F.R.I.B.A., F.S.I., M.C.I., District Surveyor of St. Margaret, etc., Westminster :—In regard to one remark, that Mr. Woodward has just made, with respect to a party wall, that question has been decided by the Divisional Court some years ago. It was in a case of mine against the Army and Navy Stores. They had a party wall between two of their buildings, and they had windows in the upper part of it. I proceeded against them, to have the openings closed up, and the magistrate decided in my favour. There was an appeal, and the superior court decided, that it ceased to be a party wall, as soon as it reached 15 in. above the lower building. The Act now reads that a party wall must be carried up 15 in. above the lowest building instead of the highest, as far as I can understand it. There is one other criticism, I should like to make about the Act, and it is that, in the London County Council (General Powers) Act, 1908, we are told in section 17, that no building or part of a building of the warehouse class shall extend to more than 250,000 cubic feet, unless divided by *division walls* in such a manner, that no portion of it shall exceed 250,000 cubic feet, and section 18 of the same Act deals with the openings in such division walls, and sections 75, 76, and 77 of the London Building Act, 1894, are repealed by those sections. We have not any definition of a division wall in any of the Acts. In the 1894 Act we are told, what a party wall is and what a cross wall is, and we are told what thicknesses an external wall and a party wall and a cross wall must be, but there is no such definition about a division wall. Section 19 of the 1908 Act says, that a division wall shall have all the characteristics of a party wall ; but then we are met with another difficulty. In the London County Council (General Powers) Act, 1909, which for shortness we call the Steel-frame Act, section 22 says, that buildings may be erected, in which the loads and stresses are transmitted to the foundations by a skeleton framework of metal, or partly by a skeleton framework of metal and partly by a party wall or party walls, and it says nothing about division walls, so when we get a new Act it appears to me, that that is a thing, which ought to be cleared

up, otherwise the lawyers will have a good time. We want to know, what a division wall is, and what thickness it must be. I suppose we may assume, it must be the same thickness as a party wall, although it does not absolutely say so. I think I understand, why "division wall" has been inserted, because a party wall is defined in section 5 (16) of the 1894 Building Act as a wall, which is used for the separation of adjoining buildings, belonging to different owners, or which stands to a greater extent than its footings upon the land of different owners, and as these party walls, as they used to be called, divide such buildings as Harrods and the Army and Navy Stores, and do not stand on the land of different owners, but all on the land of one owner, I suppose for this reason the words "division wall" have been introduced instead of the words "party wall." But when a new Act is framed I hope, that this point will be borne in mind and made clear.

MR. C. S. MEIK :—I should like to emphasize, what Mr. Woodward has said about the manufacture of reinforced concrete. It is useless to go and make very stringent provisions for the designing of work, unless provision is made for the supervision of the work, when it is being carried out. I was one of the first to take up the manufacture of reinforced concrete, and my experience has been, that more strict supervision is absolutely necessary. To give an instance, about ten years ago I carried out some work down the river, with good contractors and strict supervision, with two inspectors, one of whom was always on the works, which were not very extensive and included square columns with vertical bars, with what are termed stirrups or ties, to tie them together. The construction is familiar to every one, who knows reinforced concrete work. About two years ago some of the work had to be removed. I am not exaggerating, when I say that, for lengths of 6 ft. in the length of the column, there was not a stirrup to be seen. That is an instance of where the work may be designed with great care, but unless it is efficiently carried out the skill exercised in doing so may be useless. I have no intimate knowledge of

the Building Acts of London, but it appears to me, that the London County Council, in drawing up their regulations, have done a great deal to discourage the introduction of reinforced concrete. If they could have made them simpler and easier for the contractor or builder, I think, reinforced concrete construction would have been much more adopted, than it is at present.

Mr. LAWTON R. FORD, A.R.I.B.A., F.S.I., M.C.I., District Surveyor for St. James's, Westminster, in joining in the vote of thanks, spoke of the anomalies of the London Building Acts and gave some humorous illustrations.

As a District surveyor, he said, he was often surprised at the acquaintance of the Building Acts possessed by architects and builders, for personally he frequently made discoveries, as to how the law might be read. He argued, that a consolidated Act with the necessary amendments, doing away with the anomalies and discrepancies, was one of the pressing needs of London, pointing out that the principal Act had this year attained its majority, and that sufficient experience had now been gained, to construct one London Building Act, particularly as great changes had taken place in building construction during the past twenty-one years.

Mr. Hills had pointed out, that a domestic building included a dwelling-house and any other building not being a public building or of the warehouse class, but he (Mr. Ford) thought, that was open to misinterpretation, as a public building like a large hotel was both a domestic and a public building, and consequently needed an open space, with which Mr. Hills agreed.

Mr. Ford also sounded a note of warning to architects, as to the necessity of absolutely determining with their clients the uses, to which floors in 1909 Act buildings were to be put, and of the necessity of providing for sufficient strength for alternative uses. For example, floors of residential chambers over shops could not be used as office floors.

He also suggested, that architects should avoid employing reinforced concrete in buildings, which are

likely to be altered from time to time, to suit the exigencies of different lettings.

He frequently found, that when builders asked him to inspect bottoms of trenches for foundations, which were alleged to be in gravel, they had practically gone through the gravel, having taken out that which would have been better left in.

Mr. Ford referred to the unreasonableness of not receiving the calculations for a steel-framed building, until the builder gave the official two days' notice, when perhaps a hundred sheets of drawings of steel-work and several books of calculations had to be checked, involving perhaps the work of a month or two.

He also referred to the objectionable practice of having wide, thin plates on R.S.J.'s and ignoring the question of tack riveting, or employing such rivets too far apart. Also to the rather drastic requirement in the Act, as to rivets having to be capable of transmitting the whole of the load from the gusset pieces of a stanchion to the base plates, when perhaps the bottom of the R.S.J. stanchion is machined and bearing on the base plate.

In conclusion, he referred to the fallacy of relying upon floor concrete, to stiffen up overstressed filler joists, and to a recent interesting article in the *Architects' and Builders' Journal* upon that subject.

Mr. PERCY HUNTER, A.R.I.B.A., District Surveyor for South Lambeth :—I find that the discussion is upon a paper, which has been advocating the consolidation of the London Building Acts. Well, I remember precisely the same discussions taking place twenty-two years ago, before the 1894 Act was framed, and at that time also the cry was for consolidation. I am afraid, that the experience of the past twenty-two years will be repeated in the future, however much you consolidate your present Acts. There will be advances in building construction, and there will be the necessary legislation to deal with them, and it will necessarily ensue, as far as I can make out, that whatever Act you pass in any given year, in ten years' time you will want an Amendment Act. At the time, this discussion arose over the by-laws of the 1894

Act, I took the trouble to collect all the Acts, that I could find from the year 1666 down to 1855. They are very numerous, and somewhere in George III's time two Acts, 1774 and 1762, were passed for amending previous Building Acts. If you examine those Acts in the way I did in 1893—it is now more than twenty years ago, and I cannot carry all the details in my mind—it is curious to find, how the spirit, that seemed to actuate the people, who drafted the amending Acts, as they followed one another, was to get away from anything like definite instructions in a Building Act with regard to sizes of materials, scantlings, joists, and so on. If you turn to the 1667 Act in the time of Sir Christopher Wren, you will find, that timbers are carefully specified, and walls also are specified, and throughout all the Acts, which followed down to 1855 there seemed to be a principle involved, that you should as far as possible avoid definite figures, because, I take it, the result of such definite limitations in an Act is to take away from the individual responsibility of the architect and the builder. It seems to me, that in dealing with buildings, if the principle of responsibility for his design is taken away from the architect you might as well abolish him altogether. We have now broken away from that tendency of which I have spoken, partly in the 1894 Act, and still more so in the 1908 Act. There is very careful definition in these Acts of the materials, which should be used. This bar has only led as far as I can understand, to hopeless confusion. There has been an endeavour to devise or draw up a certain number of regulations, which shall be in conformity with the definitions or instructions, contained in the Acts, and they are so contradictory and innumerable, that people find them absolutely unworkable.

Before I finish my few remarks I should like also to refer to another matter, and that is the question of reinforced concrete construction, because during the past six or seven years I have personally had some experience of it, the King's College Hospital having been built in my district at Herne Hill. Those plans were submitted to me first in 1908, before this Act, defining all the principles of reinforced concrete construction, was brought into legislative form, and it

was a very considerable problem, which was put before me, when these plans were submitted to me. Fortunately for myself, I had my relative and friend Mr. Meik to go to ; he and his brother, Mr. P. W. Meik, gave most valuable assistance in instructing me in the principles that should guide me in criticizing the plans submitted to me. I think the meeting will agree with me, that my position was a difficult one, because under the 1894 Act the District surveyor is made responsible for the construction of public buildings as regards walls, roofs, floors, and so on, and therefore when he has to examine the designs of an architect he must be prepared, if he finds fault with the construction proposed, to give chapter and verse for the reasons of his objections. Dealing as I then had to do with this new method of construction, I felt that I could not, without expert guidance, properly perform these duties.

MR. S. BYLANDER (Past Chairman J.I.E.), M.C.I. :—I think the time has come, when the law should be amended, so that the designer should accept responsibility for the design. In practice the builder does not check the strength, and would not reasonably be expected to know the strength of a structural member in a building, erected under the Steel-frame Act or Reinforced Concrete Regulations. In the case of a brick or stone building the foreman would probably be qualified to judge roughly as to the strength of a pier or wall, but this is not the case with a more complicated method of construction. The quantity surveyor, I believe, is responsible for the accuracy of his quantities, and in the same manner I think that the designer should be responsible for the accuracy of his design. With regard to the safety of the structure, I think that in order to simplify the work of preparing the design for a building next to another building, I would suggest that for all buildings, where steelwork or reinforced concrete is used as pillars, a set of plans should be deposited with the London County Council showing the work as executed. The architects or public should have the opportunity of examining these drawings upon paying a fee, in a similar manner as the Companies' documents are now avail-

able at Somerset House. In the case of several jobs, with which I have been connected, it has been difficult to ascertain, if the load of the adjoining building has been carried on pillars or walls. If the building was erected a long time ago, it may be impossible to obtain plans, and if such were filed with the London County Council a record could be kept at a very small cost. On several occasions I have had to design a corner pier of a building next to another building, and invariably the only method, whereby one can ascertain, if the adjoining building is carried on steel or brickwork is to cut a hole. It is obvious, that if this pier is only a brick pier and supports the building, the cutting of such a hole probably reduces the strength of the pier to half, and it is not right to do so, if it was just strong enough before. It would be too weak after cutting a hole. I think, that the law should call for a penalty to be paid by the builder or sub-contractor for neglecting to carry out the work, as shown on the drawings. That suggestion would meet several of the recommendations of the speakers to-night—viz. that equal if not more stress should be laid on the correctness of carrying out the work as compared with design, and I can see no other method than asking the builder to supply and sign a certificate stating that the work has been carried out in accordance with the drawings, the certificate to be countersigned by the foreman or the gentleman responsible to the builder. I think, Mr. Hills will agree with me, that it is quite impossible for the District surveyor or for the architect and engineer to watch every bolt or rivet being put in, and that if the contractor purposely neglects to do the work correctly, as intended, it is not always certain, that the mistake is discovered. The work is covered up, the loads applied, and the building may collapse. My argument of imposing a fine is based on the well-known fact, that by a threat to touch the contractor's pocket you can best obtain satisfactory work. There seems to be a great difference between the London Regulations and the Continental Regulations. Personally I think it would be well to do away with the small internal courts, which are so usual, and have requirements, that where courts are provided they

should have an area proportionate to the site, extend down to the ground-level, and have direct access to the street. In theory as well as in practice I think, that if some of the office buildings in the City were replanned on a large scale with big courts and considerable height, more accommodation and better light could be obtained on the same plot of land. I hope, that the Town Planning Act will enable London to obtain a modified plan, with proper streets, higher buildings, and better lights. At present the maximum height is 80 feet. Two stories are allowed in the roof. I think that should be modified in this way. The maximum height of the base of the roof could be 80 ft., if you like, but as an alternative to a sloping roof the front wall may be set back and two additional stories built, provided they do not encroach upon the angle of light now prescribed. I will now suggest, what I fear will not be approved by all of you—viz. that higher buildings be built under special conditions, fire-resisting, and with metal furniture. It should be permitted in such case to build up to a greater height than at present, provided that the adjoining property has been laid out to allow for proper light. If one compares the buildings of New York with those of London, I have no hesitation in saying, that the high buildings of New York provide better light for their offices.

It would greatly simplify the preparation of drawings and plans, if the London County Council had sole control over the construction of vaults and pavements, and that regulations were made as to requirements of strength. As far as I know, there is no accepted provision for loading. Many pavement lights are decidedly weak, having most unsatisfactory supports. I do not see why constructional drawings should not be submitted for pavement lights as well as for other parts. The London County Council makes requirements as to stairs and exits, and I think it should also control the safety of the side-walk. I hope that the District Surveyors' Association will consider the advisability of supplementing the Act with a book of explanations, defining the meaning of the Act, to ensure a more uniform work. Already the District Surveyors' Association has published a small,

very ably edited, book, and I only suggest, that it should be revised and enlarged. The suggestion has been made at this Institute, that its members as well as members of the District Surveyors' Association should meet and discuss details, in order to obtain more uniform application of the Act. I would like to emphasize the following points: (1) If pillars are designed with a butt joint, the butt joint shall take the whole load and the ends of pillars shall be machined. Splice plates shall not be calculated to assist the butt joint. (2) If the butt joint is not perfect, it shall be fully spliced, and rivets shall take the total load. (3) If all holes in pillar shafts are not filled with rivets, the holes shall be deducted from the section of the shaft in the calculations; bolts cannot fill the holes. (4) Bolts and rivets in the same group or connection cannot be assumed to act together; either bolts or rivets should be used. (5) Bolts and bolted connections carrying heavy loads must not be too short; they must be long enough, to ensure that the bearing is not applied on the thread, and they should be fitted—that is, turned bolts should be used with small clearance in holes reamered in position. I have seen so many unsatisfactory cases of this, and in each case I have felt, it was most serious. The holes are not in the true position; they are not opposite, and only a few of the bolts have to carry the load, instead of the whole number. (6) Bases to pillars should be so made, as to transmit the load properly to the grillage beams. The gusset pieces and base plate should be of adequate strength. (7) The diameter of the rivet plus $\frac{1}{8}$ in. should be deducted from the gross area, to obtain the net area. Holes are usually $\frac{1}{16}$ in. more than the diameter of the rivet, and are often not perfect and must be reamered up for a larger rivet.

MR. GREEN, in the course of his remarks, shewed that a deflection of $\frac{1}{400}$th of span, working with an elastic modulus of 13,400 tons per square inch, is practically equivalent to $\frac{1}{360}$th of span working with the much more convenient figure 12,000 for the elastic modulus, which figure makes an allowance for the deflection due to shear. With regard to columns

with fixed ends, he deprecated the practice of District surveyors in regarding a fixed end as an unrealizable ideal. He understood from Mr. Huddlestone, that in devising a formula for columns, that had not both ends fixed, the allowable stresses were made low, so as to discourage the use of such columns.

Mr. E. FIANDER ETCHELLS, Assoc.M.Inst.C.E., A.M.I.Mech.E., F.Phys.Soc. (Member of Council C.I.) :—At this late hour it is impossible for me to go at all fully into a tithe of the matters, that have been raised this evening, and therefore I will confine myself to one or two, which perhaps may slip from your minds, if the answer is deferred until some later occasion.

Hinged Ends.—The question of hinged ends has been raised, and a certain eminent architect explained to you this evening, that he is not aware of the intention of the term. The Draft Regulations for Reinforced Concrete are later than the Steel-frame Act, and whilst I am quite aware that regulations cannot in any sense be taken as overruling an Act, yet they do as a matter of fact give some clue as to the mind of the building authority. Where in the Steel-frame Act you find "hinged ends," in the Draft Regulations you see that term replaced by the phrase, "End fixed in position but not in direction." The term "fixed end" you find replaced by the term, "end fixed in position and direction." That is also supplemented by a clause defining fixed ends: "A pillar or other structure shall be deemed to have fixed ends, when the ends are sufficiently secured to other parts of the construction, having such rigidity as will maintain the axis at the ends in its original position and direction under all loads less than the crippling load."

Elastic Modulus.—With regard to the question of the elastic modulus for steel—12,000 tons per square inch *versus* 13,400 tons—it has been stated that the figure was hidden away in the archives of the Council. Perhaps it is written down there, but it has also been sent to all the technical institutions, and it is written plainly, that the elastic modulus for steel shall be taken at 30,000,000 lbs. per square inch. That figure is to be found in the Regulations for Reinforced

Concrete. You must not expect, that any building authority would be wantonly inconsistent and stipulate that one elastic modulus for steel shall apply for a steel complying with the British Standard Specification and another elastic modulus shall apply for the same steel complying with the same specification. So there is no doubt as to what figure may be taken for the elastic modulus of steel in the Steel-frame Act.

Anomalies.—The other points, I wish to raise will not detain you more than a moment or two. The first is, that there is a wish on the part of all of us to remove anomalies, but I caution you that anomalies cannot be done away with; you merely drop one lot and drop into another lot. You can only hope, that the anomalies to come will not be so confusing and so distracting as some of the anomalies, which our legal friends have already laid in store for us.

Suggestions.—I also wish to announce, that the Science Committee has already taken steps to call meetings to follow the suggestions raised by Mr. Hills' paper, and also by the papers of Mr. Harold Cane on Brick Chimneys, of Mr. Cocking on Steelwork, and of Mr. Bylander on Steel-frame Buildings. Any member of this Institute is entitled to make any representations on any points, which he thinks should be considered, not to the County Council nor to the Board, because there has been no intimation that they are for the moment prepared to amend the Acts. As a matter of fact, both authorities know, that amendment is required, and that fact is manifested by the endeavour to amend the Act prior to 1905. The Council, at any rate, was willing at least ten years ago, but the technical institutions opposed the amendments.

Ideal Building Law.—With regard to building laws in general, the ideal building law is that, which is most in accordance with natural law, and by natural law I include mechanics, and hygiene, and ethics, and also the true laws of sound finance in the highest acceptation of that term.

Deflection of Steel Beams.—With regard to the question of deflection, I can give you a simple formula, which will avoid you worrying, as to what the elastic

modulus might be. The rule is, that the inertia moment (for a steel joist or a steel girder required to comply with the Act of 1909 in respect of deflection) is equal to the weight in tons multiplied by the square of the length in feet, divided by 17·9.[1] If you use that rule, you will find you have a girder, which complies with the limit of deflection. You need not use that rule, if the span does not exceed twenty-four times the depth of the joist.

Punctuation.—I am sorry Mr. Woodward is not here, because I could have shown him the importance of a comma, and I could have brought it home to him by these two phrases. You will see that the same words are repeated: "Mr. Woodward said the lawyer was mistaken." "Mr. Woodward, said the lawyer, was mistaken." Two commas have been inserted in the latter case, and they have reversed the meaning.

MR. M. NOEL RIDLEY, M.Inst.C.E., M.C.I., in a written contribution, said:—I consider that the Acts should be consolidated into one Act, so as to cover the ground without contradiction and overlapping.

There are only three kinds of buildings—viz. framed, partially framed, and non-framed buildings. A reinforced concrete building and a steel building may be both framed, and, further, steel and reinforced concrete can be, and often are, combined so as together to form a framed building. By having three main Acts, as at present, difficulties arise, which could be so easily avoided with one Act, and so smoother working would be obtainable.

I shall now mention a few detail points that require remedying:—

Inaccurate moments should be omitted, as they affect the point of contraflexure. Complicated formulæ should only be given as illustrations; any satisfactory formula, the designer prefers to adopt, should be permitted.

Details of work, such as pillars in clauses 83 to 88, should be treated in the same way.

No type of reinforced concrete construction, known

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[1] $I = \frac{W L^2}{17·9}$ or $I = \frac{W L^2}{17·9} \times \frac{\text{in.}^4}{\text{tons} \times \text{ft.}^2}$

or unknown, should be excluded, if found to be satisfactory. For instance, the minimum thickness of walls is given as 4 in., but the well-known dovetailed corrugated steel sheeting wall, which has been on the market for about eighteen years and is still reckoned one of the strongest, is for outside walls practically barred. This type of wall is used for outside work, and in many cases 2 in. thick is absolutely satisfactory.

The object of these Acts is not to restrain progress in building construction, but to protect the public; in many cases the old Building Acts have not done this.

No Building Act should be so framed, that it cannot easily be altered. To get over this difficulty, I suggest, that the Act should have few clauses and only of such a nature, that alteration is undesirable.

Provision should be made for the London County Council to add an addendum, which can be quickly altered or added to, as circumstances may dictate.

All details of construction, formulæ, etc., should be placed in this addendum. If a type of building construction is desired to be used and is excluded from being used, provision should be made for applying to the Council, and if such construction is found to be satisfactory then permission to adopt it should appear in the addendum.

By some such means the new London Building Act would obtain to an authoritative position, and so become the standard for the United Kingdom, and not receive the strong criticism that is meted out to the Reinforced Concrete Act.

MR. THOMAS P. TINSLAY, M.C.I., in a written contribution, said:—There are one or two remarks that I should like to make in relation to Mr. Hills' paper.

The points I would refer to are:—

On page 74, third paragraph, Projections, Mr. Hills states that these must be of *fire-resisting* materials. By section 73 (1) of the 1894 Act they must be of *fireproof* material. In the 1905 Act there is a schedule of fire-resisting materials, but under section 73 (1) of the 1894 Act projections must be of brick, tile, stone, artificial stone, slate, cement, or other *fireproof*

material. What is fireproof material? I know of no schedule of such materials. This is, I think, a case in which the Act desires amending when the time arrives.

On page 75, paragraph 6, the term "wholly in one occupation." The interpretation given by Mr. Hills of the meaning of this term is not, I think, the proper one. The term, in my opinion, means actual occupation of the whole of the premises by one person or firm.

On page 81, at the bottom of the page, it says, "the builder, etc., is required to pay fees . . . on (1) every new building, (2) on every addition or alteration." Fees are also payable on "other works," as well as on additions or alterations.

On page 83, at the top, Mr. Hills says that the London County Council has only availed itself of making five by-laws. This is not so, as I would mention these by-laws were made under section 16 of the Metropolis Management and Building Acts (Amendment) Act, 1878, and section 12 of the London County Council (General Powers) Act, 1890, and not under Part XIV of the 1894 Act. These are still in force, and are confirmed by section 216 under Part XVI of the 1894 Act.

On page 87, paragraph marked (1) (*b*), in addition to a building in which accommodation is provided for more than twenty persons this Act of 1905 is also applicable to a building "constructed" or "adapted" to be occupied by or "constructed" or "adapted" for the employment of more than twenty persons.

On page 88, top line, in addition to all existing buildings all new buildings exceeding 30 ft. in height must be provided with means of access to the roof.

On page 94, third and fourth paragraphs. Buildings can only be erected to a height of 80 ft. if the street was formed or laid out before the 7th August, 1862, as by section 49 of the 1894 Act existing buildings cannot without special sanction be raised or new buildings erected on the side of a street laid out after this date to a greater height than the width of the street.

Taking Mr. Hills's illustration on page 95 (Fig. 1).

If this street was formed after August 1862, the new building, I take it, could not without special consent exceed 13 ft. in height, irrespective of whether the plans of the old buildings had been certified or not.

MR. JOHN W. GRANT, A.M.C.I., A.F.P.W.Inst. Lond., in a written contribution, said :—There is one point I should like to draw attention to, and that is, in designing large public buildings close to the line of main thoroughfares too frequently some architects in Scotland are in the habit of having a heavy projecting cornice running usually, say, 3 or 4 ft. above the pavement just at the level of the window-sills of the ground floor. This projection is very often destroyed, as it is tempting to mischievous boys. Probably it may be difficult in making the design to get rid of this ornamentation, but it is a matter that should be kept before the architect, as I know of several instances where the projection had to be dressed flush. Perhaps this is beyond the power of the Act to interfere.

SIXTY-SECOND ORDINARY GENERAL MEETING

THURSDAY, MAY 6, 1915

THE SIXTY-SECOND ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, May 6, 1915, at 8 p.m., at the conclusion of the Sixth Annual General Meeting of the Institute,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

THE PRESIDENT :—The first business is the further adjourned discussion upon Mr. Osborn C. Hills' paper, entitled "The London Building Acts, with some suggested Amendments," and I will call upon Mr. Allan Graham to resume the discussion.

DISCUSSION

MR. ALLAN GRAHAM, A.R.I.B.A., M.C.I. :—I hope, the fruition of this discussion will not be greater stringency, but rather greater leniency, in connection with the Act. Many of us remember the inception of the 1894 Act, and how for a year or two later even the District surveyors differed and were not quite sure as to its interpretation. Generally I found that the surveyor with architectural training took more reasonable views than those with a bureaucratic training; for that reason I hold, that every one, who attempts to alter or deal with the London Building Act ought to

have had architectural experience, to understand the tremendous difficulties, that an architect has to face in complying with all the regulations with regard to open space, etc., in a city like London, where ground is so expensive and where the regulations tie one up. I would prefer to have the London Building Act of 1894 remain as it is, except the proposed alterations operate in the direction of greater leniency and freedom. I consider, that the suggestions of Mr. Hills, with the single exception of allowing flues to be 2 in. from the centre of a party wall, are all in the direction of greater stringency. I will draw attention to some of the points, he has raised. With regard to the vestry case, which he adduces as being a great hardship to the District surveyor in the matter of fees, I would say that, if he is going to improve the Act in respect to the District surveyors' fees, I would ask him also to put in safeguards on behalf of the owner, as I know of cases where the boot has been on the other leg. Two great blocks of flats about nine stories high were connected at the basement with a roofed-in passage, 6 ft. broad by 8 ft. high, and the fees for the District surveyor were calculated as an alteration to two blocks of flats and amounted to £27 for a structure, that would not cost £8 to build. If he is going to insist on getting fees for an alteration of a church added to a vestry, then I must insist, on the other hand, that provision should be made to meet such a case as I have stated. True, the District surveyor always puts a provision, that if this fee is paid within fourteen days from date a sum of so much will be accepted, but——

Mr. W. G. PERKINS, District Surveyor for Holborn :—Was the full fee charged in the case you mentioned?

Mr. GRAHAM :—No. Half was charged, but £13 10s. for such a structure is a bit too heavy. Then, again, I see, Mr. Hills has introduced a diagram on page 95 with regard to certifying buildings under section 13. This is a case, where the street is 13 ft. wide, and it is possible under the 1894 Act to carry a building up 80 ft., plus two stories in the roof, as he says. I could not agree to his suggestion, for

if he applied his reasoning to a building on a site, 40 or 50 ft. wide, when 10 ft. has been allowed in the rear and then a $63\frac{1}{2}$ degree diagonal line has been drawn 16 ft. up, as required by section 41, the building would be cut off at the fifth floor, and then only a box-room at that, and no stories in the roof. This would be an unfair restriction in places like Westminster and the neighbourhood, where land is very expensive and costs anything from 4s. 6d. to 7s. 6d. a foot per annum; people in that neighbourhood are not philanthropists, and cannot be expected to give 13 ft. 6 in. depth of site away; where no consideration would be given, I consider, that it is really asking too much of the long-suffering owner. If he were willing to give a quid pro quo in the rear as compensation for loss at front, then the proposal would be reasonable. He wants to add another to our difficulties—viz. that in all these common houses, that have trimmer hearths to protect the floor joists against fire, he wishes us to support them with some steelwork. I hold, that the function of a trimmer hearth is served, if it prevents the house from taking fire. Surely firemen know by this time, that it is dangerous to stand under a trimmer hearth, supported by wooden joists. Therefore I do not support that proposal. Under section 59 Mr. Hills wants to raise party walls 3 ft. above any buildings within 5 ft. I consider, that that, again, is a provision which is hardly necessary, because even then it would be 5 ft. interference with the light of his neighbour, and for that reason alone would be objectionable. With regard to the ventilation of staircases, he would like to bring it under the "courts within a building." It is difficult enough at present for an architect to get anything like the value out of his ground. A better disposition of rooms is likely to be got, if the staircase position is left to the judgment of the architect as at present, and, after all, a staircase is not a habitable room, and most of the buildings now have lifts and the people scarcely use the staircase. On page 100 one will quite see the tendency of regulations. There is an elevation by Mr. Hills, which shows very successfully the operation of an angle under section 84 (2) of the Act of 1905. There is a great portion of that wall, in which

a window could not be placed. That is absolutely precluding ventilation in a back wall all for the sake of the prevention of the spread of fire from window A to the back wall. That might be obviated by putting a bit of wired glass in the windows. I speak feelingly in these matters, for designing a building in London is different from designing one in Scotland. In London you have not merely got these regulations to contend with, but all round about you bristles with Ancient lights, and you can scarcely build a wall without some person opposite you trying to prevent you, and that operates very hardly at times, when your site has been unbuilt upon and your neighbour has built adjoining windows overlooking your site. Without obstruction for nineteen years and a day they become Ancient lights, and you have to give him about 45 degrees of light over your site. With that operating against you, and all these other regulations, you will see, it means that you cannot put up a building in London, to give a reasonable return. The tendency of regulations is to get worse and worse, because when committees sit, each member has his own special fad, and it ends by evolving the greatest common measure of fads! It is not so bad, when one leading mind sits down to a thing, because he brings reason on all the points, but when many confer together nothing will satisfy them but that every point suggested, shall be incorporated. I was at the R.I.B.A. a fortnight ago, and this very point arose with regard to sections 74, 75, and 76 with regard to the division of buildings. I may say, that in the 1894 Act these were repealed and another Act was substituted for them. The Act substituted was even worse than the original Act, and so much so that Sir William Lever, who is looked upon as a pioneer in factory buildings, tried to build some factories in London, but this division of 250,000 cubic feet became so harsh, that in a high building it meant, that party walls had to be built every 10 ft. Such a regulation made economical control impossible. He and Mr. Max Clarke stated, that these regulations were absolutely impossible and against factory buildings being carried out in London, and it was pointed out that, although many people look at Lancashire and the Midlands as the

great manufacturing centre, yet London is the greatest manufacturing city, and the regulations ought to be a little milder. I might point out, that even in Glasgow the building regulations are considerably milder, because in the same class that we allow 250,000 cubic feet for the division, Glasgow allows 350,000, or 100,000 ft. more, and it is even said, that you can have 750,000 for a weaving shed, or if your building has existed before, you are allowed 900,000 cubic feet. If with 900,000 cubic feet there is not a danger from fire, then it ought to be extended to other buildings which require the extra space and in which supervision is necessary from a central position. That is my belief, and also I have often thought that all these regulations in London tend to kill the architect entirely. In Glasgow or the North of England all you have to do, when you have a set of plans, is to put your complete plans before the Dean of Guild authorities, and you have no more trouble about it, whereas here not only have you to place the plans before the District surveyor, but you have to place with the local councils a set of your drawings for the drains and also a set of fire escape plans with the London County Council department. Some people say, you can get the money out of your clients, but they are cleverer than I am. As a rule clients expect 5 per cent. to cover everything, and to get anything extra is like getting blood out of a stone, so that I wish the regulations here were more in accordance with those in Glasgow. If we had one authority in London, as in Glasgow, the architect would come again into his own and not be harassed with all these various troubles, which touch him in the exercise of his practice here. Again, in Glasgow all charges are borne by the Corporation; the client has not to pay for the passing of his plans at all. Mr. Hills has, I think, mentioned the fact, that as the plans were for the building owners they ought to pay, but the argument cuts both ways. When people build in towns they help to pay the rates, and anything in the nature of a building in a town is to the town's advantage.

Mr. W. F. Christmas, M.C.I.:—May I confine myself to the last section, that known as the 1909

Amendment? It would be an advantage, if the table of superimposed loads on floors could be extended. Many purposes, for which floors in buildings are used, are not covered, and it is very often difficult to determine under which class to design. A building may be a building of the warehouse class for other reasons than that it is a warehouse and thus subjected to heavy floor loads, and a load of 224 lbs. may be demanded in a building, to be used for very light purposes.

District surveyors should have power to inspect frame buildings at reasonable times, after their erection, to enable them to see that the ordinary use of the floors is not in excess of their designed loads. There is nothing at present to prevent an owner or lessee doing whatever he likes with his floors, however strong or weak they may be.

Why cannot there be a reduction allowance for columns in warehouse buildings, as in office buildings? The floor area cannot be loaded fully, there must be means of access, and, assuming the proper figure is adopted to begin with, surely the greater the superload the greater the allowable percentage of reduction. The behaviour of foundations may be a serious matter here. Thus columns with a large proportion of actual load—i.e. brickwork and small floor—and columns with all floor load would have the same area of foundation, but the first would probably settle considerably more than the second and may cause serious trouble. It may be even possible to design floor slabs and secondary beams to carry the full load, primary beams providing an allowance off this, and columns a still greater percentage of reduction.

Considering riveted joints of pillars and their bases, this is a matter involving considerable difference of opinion. If the principle of contact is allowed, as it obviously is in joints, why put the whole load through rivets into gussets? With machine-faced ends of pillars and faced surfaces to steel grid foundations no rivets at all may be required, except for purposes of bolting down, when the width of pillar is the same as, or even less than the width of such grid. Of course, with larger loads involving large grids, say 400 tons and over, specially designed bases are necessary, but

then only a portion of the load need go through rivets. The stress allowed in foundations under purely static loads might perhaps be increased, and greater discretionary power given to District surveyors in their administration would, I think, make for efficiency and smooth working of this highly technical part of the London Building Acts.

MR. EWART S. ANDREWS, B.Sc., M.C.I. :—I suggest, that you should not specify the projection beyond the beam, but the projection beyond the centre of the rivet. It is the question of the unsupported thin piece of plate, which of course will be subjected to a secondary buckling effect and will not carry anything like its proportion of the load. I have a note on "end fixing." That is a subject about which I have worried quite a lot. It seems to me, that one of the chief difficulties in this matter is the manner, in which the allowable stresses have been specified in the Act, apparently intentionally to drive you to build with your stanchion as one with fixed ends. There is, I believe, no scientific justification whatever for the stress for the hinged end being reduced to the large extent, that it is on comparatively short lengths. All of the investigations and formulæ for columns—and they are legion—I believe agree that the safe stresses should approach to a common value in a very short column, and the effect of the great reduction in the allowable stresses on the hinged end under the Act is to drive people to have, what they call, fixed ends. If, on the other hand, we were not driven in that direction, and the figures were made more lenient and more in accordance with fact, I believe the difficulty would be to a very large extent solved. That brings me to the question of the rivets in the base. Of course the number of rivets connecting the base plate to the main column has really to do with the question of the fixity. If an end is perfectly fixed, it will have to bear a very large bending moment, before buckling can take place. The first effect of a weakly riveted end would be, that the rivets would give and the end would then behave as hinged, so that it is not only a question of transmitting the pressure direct, but it is a question of

carrying the bending moment, which would be induced directly buckling started to take place. Nearly all the experiments, which have been carried out on large size built-up sections have come to the result, that what we ordinarily call the fixed end—that is to say, a fairly well-riveted end—does not behave as fixed in a test, and should really be designed as hinged. The penalty, which the Act gives, really prevents you from doing that, but if the figures were modified in the manner which I suggest I think that would help the matter very much. In regard to the load on floors, I agree with the suggestion, that 100 lbs. per square foot is rather a high figure for office buildings, and I think the difficulty would be met, if the rule specified not only a superload but also a concentrated load. You can design either for a certain spread load or a concentrated load ; and if any revision is made of these floor loads, might I suggest that the 112 and the 224 lbs. should be dropped? They are relics of the cwt., and the sooner we get rid of the cwt. the better, I think, for all concerned. Just one further point. In the very interesting speech of the last speaker he referred to columns and the fact, that all the bays of a heavy warehouse floor could not be loaded together. It might be more serious to have some bays loaded and some unloaded, than to have them all loaded together, because they produce an eccentricity of load on the columns. Although extremely difficult to investigate satisfactorily, as far as investigations have gone I think they have shown, that that is more serious than having the whole of the floor covered.

MR. H. KEMPTON DYSON (Secretary C.I.) :—Ten years ago in a leader in the *Builders' Journal* for April 5, 1905, I wrote : " It is important that the present Acts should be revised, and we consider the Council will be well advised not to wait longer than next year. We hope no half-measures will be attempted, but that the old Acts will be repealed and a comprehensive new Act brought in, because the present Acts and amendments are quite confusing enough to professional men ; a building ordinance should be businesslike in form. Hitherto the sugges-

tions of professional bodies have been treated with scant courtesy. After receiving suggestions the Building Act Committee should send their altered clauses for further suggestions, and not try to rush through a Bill in defiance of all interested bodies."

Now the procedure adopted with, and the form given to the proposed Reinforced Concrete Regulations conform most closely to my old ideals. The co-operation of the professional societies has been sought, and their recommendations treated with courtesy; the altered proposals have been sent again and again for further suggestions, while the fact that they are to be Regulations and not an Act of Parliament will permit of their being revised and kept up to date. The language is straightforward and more intelligible than of old; but I still await that desired complete overhauling of the existing Building Acts. The changes within recent years in the outlook on methods of constructing buildings point clearly to the advisability of having the building ordinance in a form, which can be easily modified from time to time, and it seems to me, that Regulations are the elastic form required. We have a Code Napoléon for our building law, and if all legal precedent for this country is against it, then let such matters of general policy as the width of streets, lines of frontage and rights of adjoining owners, and space about the rear of buildings be enacted, but as regards the details of construction, let them be given in regulations that can be modified from time to time. The present Acts are not only very disorderly and involved in expression, but are most unscientific. The Steel-frame Act and the Reinforced Concrete Regulations have specified stresses, methods of calculation and workmanship, and it seems to me, that the same ought to be done for the other materials and constructional methods in vogue (such as timber-work, stonework, brickwork), and some which may yet be in vogue (such as reinforced brickwork, reinforced stonework, reinforced steel, reinforced glass, reinforced papier mâché, nickel steelwork, aluminium alloy-work, concrete slabs and blocks for hollow and solid walls, and so on). The regulation of the thickness of walls by the height of the story is out of date in this scientific age; the

thickness of walls should be determined by the work required thereof as to stability, resistance to fire, and hygienic qualities, such as damp-proofness. The health of the occupants of buildings is not thoroughly studied at present, for the proper number of changes of air per hour are not regulated, while unnecessary restrictions are placed upon designers, who wish to give the maximum amount of light. Among other reforms, I should like to see greater facilities given for enabling economy to be effected in the construction of foundations of buildings. The maximum load, that the ground will safely sustain should be put upon it, and footings could often be eliminated. It is important to know the character of the subsoil in the preliminary stages of the design of a building, and there is probably a mass of information on the subject in the possession of the London County Council's various departments and of the District surveyors of London, which ought to be codified and made easy of reference by the public, who might well be asked to pay for the work of codification, for it would save the expenditure of many thousands of pounds a year on building operations.

Mr. OSBORN C. HILLS, F.R.I.B.A., M.C.I., District Surveyor for the Strand :—I am sure you will not expect me to reply to all the points raised in the discussion. I am greatly obliged to Mr. Etchells for taking quite half of them off my hands. Our President in his remarks quickly dealt with one of the conundrums I asked, as to the extent, to which plates should be allowed to project beyond the rolled steel section. He suggested eight times the thickness of the thinnest plate, and if we add to this Mr. Green's suggestion of measuring the projection from the centre of the rivets, I think that is a very good working solution, but my point was that there is nothing in the Act to allow me to apply that. I cannot enforce it, and I hope, that when we get a new Act that will be included. A great deal was said about clay foundations, with all of which I agree, but I must skip what I was going to say.

A definition of building is very much wanted, but I think Mr. Perkins was right when he said, that if

we took the decisions that have been given—and he quoted one of Mr. Justice Byles'—we should not need a definition. When we do not agree with the owner we go before a magistrate, who can put us right, and I have never had any difficulty on the point of whether a building is or is not a building. Sometimes there is a question, as to whether a thing is a building or a structure, and I drew attention to that in my paper. You will note the careful omission of the word "structure" in one part of section 82, and you will see that it is quite intelligible. Mr. Perkins did not agree with me about the exemptions in the City of London. He thinks the Building Act should apply universally, but when we turn to the 1905 Bill, the London County Council there proposes not to exempt the City of London and to take powers to make every street 75 ft. wide. Look at Watling Street! If you widened that to 75 ft., you would have no Watling Street at all. You would be in Cheapside, and you would simply do away with Watling Street altogether. I do not think, that the rules that apply to the suburbs could well be brought into the City in the same way, and I think it is quite reasonable, that an ancient city, like the City of London, should be exempt from some of those rules, that are very applicable to other parts. In regard to open spaces about office buildings, Mr. Perkins thought, they should be put under the same rules as govern two-story buildings farther out. I think that is too drastic. Space is so valuable, and it is not necessary to have the same amount of open space for an office as for a dwelling-house. That only shows, that even District surveyors can form different opinions and do not always agree.

The object of my paper has been largely met by the valuable discussion which it has brought out, and although I do not think, that this Institute can initiate a new Act (nor do I think there is the smallest chance of a new Bill being framed during the continuance of this terrible war), still, we can prepare for that event, and a careful record has been made of every suggestion, that has been raised in these discussions. It only remains for me to thank you very heartily for the kindly consideration you have shown me in listening to so long a paper.

SIXTH ANNUAL GENERAL MEETING

THURSDAY, MAY 6, 1915

THE SIXTH ANNUAL GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, May 6, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

THE SECRETARY (MR. H. KEMPTON DYSON) read the notice convening the meeting and the report of the Auditors on the balance-sheet.

REPORT OF COUNCIL FOR 1914–15 SESSION

The Concrete Institute had on April 30, 1915, 935 Members, 40 Associate Members, 8 Associates, 64 Students, 5 Special Subscribers, and 14 Honorary Members.

The increase in Membership since the previous figures were given in the Report for the 1913-14 Session is shown in the following table :—

NUMBER OF MEMBERS.

	End of Dec., 1913.	End of Dec., 1914.	April 30th, 1915.
Members	948	937	935
Associate Members	3	32	40
Associates	3	6	8
Students	47	56	64
Special Subscribers	7	5	5
Hon. Members	16	16	14
Total Membership	1,024	1,052	1,066

Of this total 364 reside in London and its environs, 398 reside in the provinces, and 304 abroad.

It is a matter for congratulation that there has not been considerable decrease in Membership owing to the war, although a number of Members of the Institute are serving with the Forces. Names of Members serving with the Forces of whom information has been received are recorded in a Roll of Honour which is given as an appendix to this Report.

The finances of the Institute are shown by the accompanying Balance Sheet. In 1913 there was a deficit. In 1914 there has again been a deficit, but a smaller one, owing to an appeal by the former President, Mr. E. P. Wells, suggesting that Members should increase their subscription to the new rate of two guineas which was adopted when the total Membership of all classes had reached one thousand. The new subscription rates and entrance fees have also improved the income. By reason of the printing of the TRANSACTIONS having been delayed, a reserve on account of such printing, which is properly chargeable to 1914, has been made in the Balance Sheet. It is hoped that when peace is declared and business has resumed its normal course, the past rate of growth of the Institute in Members and income will be resumed.

The extra number of general meetings has been maintained, but in view of a number of the Junior Members joining the Forces, the Educational Lectures and the Informal Meetings of Junior Members were abandoned for this Session; such educational work will be resumed in the future.

In conformity with other technical societies, it has been decided not to hold an Annual Dinner this year.

The following is a list of Meetings during the Session :—

1914.

Thursday, November 19th. Fifty-first Ordinary General Meeting.
Presidential Address by Professor Henry Adams, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc.

Thursday, December 3rd. Fifty-second Ordinary General Meeting.
Paper by Mr. H. Kempton Dyson (Secretary), on "Shear and Problems Arising Therefrom."

Thursday, December 17. Fifty-third Ordinary General Meeting.
Adjourned Discussion on Mr. Dyson's paper.

1915.

Thursday, January 7th. Fifty-fourth Ordinary General Meeting.
Paper by Mr. Henry J. Tingle, M.Inst.C.E. (Member), on "The Application of Concrete in Modern Sanitation."

Thursday, January 21st. Fifty-fifth Ordinary General Meeting.
Paper by Mr. Ewart S. Andrews, B.Sc. (Member), on "Some Modern Methods of Arch Calculation."

Thursday, February 4th. Fifty-sixth Ordinary General Meeting.
Discussion on Draft Report of the Science and Reinforced Concrete Practice Standing Committees on "A Standard Specification for Reinforced Concrete."

1915.

Thursday, February 18. Fifty-seventh Ordinary General Meeting.

Paper by Mr. T. A. Watson, Assoc.M.Inst.C.E. (Member), on "Economy in Reinforced Concrete Construction."

Thursday, March 4th. Fifty-eighth Ordinary General Meeting.

Paper by Mr. R. Graham Keevill, A.M.I.Mech.E. (Member), entitled "Some Notes on Wind Pressure."

Thursday, March 18th. Fifty-ninth Ordinary General Meeting.

Paper by Mr. Osborn C. Hills, F.R.I.B.A. (Member), District Surveyor for the Strand, on "The London Building Acts, with some Suggested Amendments."

Thursday, April 8th. Sixtieth Ordinary General Meeting.

Adjourned Discussion on Mr. Hills's Paper.

Thursday, April 22nd. Sixty-first Ordinary General Meeting.

Paper by Mr. F. E. Wentworth-Sheilds, M.Inst.C.E. (Vice-President), on "The Design of Quay Walls."

Thursday, May 6th. Sixth Annual General Meeting and Sixty-second Ordinary General Meeting.

Report of Council. Annual Election of Members of Council. Award of Institute Medal. Further Adjourned Discussion on Mr. Hills's Paper.

Thursday, May 22nd. Sixty-third Ordinary General Meeting.

Paper by Mr. E. A. W. Phillips, M.Inst.C.E. (Member), on "Lime Concrete in the East."

The thanks of the Institute are due and are hereby tendered to the authors of papers.

The attendance at meetings during the past Session has been well maintained.

As the result of a ballot among Members of Council,

the bronze medal for the best paper read in the 1913-14 Session has been awarded to Mr. W. Cyril Cocking for the paper entitled "Calculations and Details for Steel-frame Buildings from the Draughtsman's Standpoint."

In the previous Annual Report reference was made to the enlargement of the scope of the Institute and the alterations to the Rules, which were approved at General Meetings. It was explained that alternative policies would be placed before a General Meeting for decision. This has been deferred until the war is concluded.

As regards the work of the Council and Committees during the past year the chief business has been the further consideration of the Regulations proposed to be made under the provision of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete. A revised draft of the proposed Regulations was received in September, and the Council and Committees in joint Session reconsidered the matter, and made various recommendations for further amendment. It was found that the Institute's previous suggestions had been embodied in large part in the revised draft.

A letter was addressed by the Quantity Surveyors' Association to the Commissioners of H.M. Office of Works and Public Buildings deprecating the preparation, by the Consulting Engineers (employed by the department to make calculations and prepare drawings for reinforced concrete structures) of Bills of Quantities upon which competing contractors based their tenders, and advocating the extension to such work of the present system, under which quantities for buildings erected in other materials were prepared by surveyors appointed by the department. The Council of the Concrete Institute, together with the Quantity Surveyors' Committee of the Surveyors' Institution, gave its support. The letter inquired whether a deputation from the three bodies in support of this suggestion would be received, but nothing has yet resulted.

In June 1914 Mr. J. S. E. de Vesian and Dr. J. S. Owens were co-opted as Members of Council.

The Committees appointed by the Council for the Session were as follows :—

THE FINANCE AND GENERAL PURPOSES COMMITTEE.

Chairman—Mr. H. D. Searles-Wood.

Ordinary Members—Professor Henry Adams, Mr. J. Ernest Franck, Mr. Charles F. Marsh, Sir Henry Tanner, Mr. H. J. Tingle, Mr. E. P. Wells, and Mr. G. C. Workman.

Also *ex-officio* the President and the Chairman of each Standing Committee.

THE SCIENCE STANDING COMMITTEE.

Chairman—Mr. E. Fiander Etchells.

Vice-Chairman—Mr. H. D. Searles-Wood.

Hon. Secretary—Mr. W. G. Perkins.

Ordinary Members—Professor Henry Adams, Mr. Ewart S. Andrews, Mr. H. K. G. Bamber, Mr. D. B. Butler, Mr. J. E. Franck, Mr. Charles F. Marsh, Dr. J. S. Owens, Mr. R. W. Vawdrey, Mr. E. P. Wells, Mr. F. E. Wentworth-Sheilds, and Mr. Morgan E. Yeatman.

THE REINFORCED CONCRETE PRACTICE STANDING COMMITTEE.

Chairman—Mr. S. Bylander.

Vice-Chairman—Mr. G. C. Workman.

Hon. Secretary—Mr. R. W. Vawdrey.

Ordinary Members—Professor Henry Adams, Mr. Ewart S. Andrews, Mr. Percy J. Black, Mr. F. Bradford, Mr. J. F. Butler, Mr. Alexander Drew, Mr. Oscar Faber, Mr. Moritz Kahn, Mr. P. W. Leslie, Mr. J. Petrie, Mr. F. Purton, Mr. Lewis H. Rugg, Mr. A. Alban H. Scott, Mr. H. D. Searles-Wood, Mr. T. B. Shore, Mr. B. Taylor, Mr. John M. Theobald, Mr. T. A. Watson, Mr. E. P. Wells, Mr. Morgan E. Yeatman.

THE TESTS STANDING COMMITTEE.

Chairman—Mr. A. Alban H. Scott.

Vice-Chairman—Mr. R. H. Harry Stanger.

INCOME AND EXPENDITURE ACCOUNT.

Year Ending December 31, 1914.

Dr. | *Cr.*

INCOME.

	£ s. d.	£ s. d.
To Annual Subscriptions	984 15 0	
„ Special Subscriptions...	26 5 0	
„ Students' Subscriptions	21 10 6	
		1,032 10 6
„ Entrance Fees		58 16 0
„ Sundry Receipts :—		
Various Receipts	8 12 4	
Interest on Deposit	4 19 8	
Advertisements	36 5 3	
Donations	2 2 0	
		51 19 3
„ Balance carried down		127 16 6
		£1,271 2 3

EXPENDITURE.

	£ s. d.	£ s. d.
By Office Salaries	381 0 0	
„ Office Rent, Hire of Hall, Cleaning, Lighting, and Heating	155 13 9	
„ Printing, Reporting, &c.	501 5 5	
„ Postages, Telegrams, &c.	76 4 9	
„ General Expenses	72 13 8	
„ Accountancy	8 8 0	
„ Stationery	57 11 5	
„ Books and Utensils	2 18 5	
		1,255 15 5
„ Amount written off Furniture for Depreciation		15 6 10
		£1,271 2 3

	£ s. d.
To Balance, being excess of Expenditure over Income 12 months to date, transferred to Surplus Account	127 16 6
	£127 16 6

	£ s. d.
By Balance brought down	127 16 6
	£127 16 6

BALANCE SHEET.

Year Ending December 31, 1914.

Dr. | *Cr.*

LIABILITIES.

	£	s.	d.
To Subscriptions received in advance	48	6	0
,, Current Liabilities	335	17	3
	£384	3	3

ASSETS.

	£	s.	d.	£	s.	d.
By FURNITURE:—						
As at December 31, 1913	153	8	10			
Less 10% Depreciation written off to Income and Expenditure Account...	15	6	10			
				138	2	0
,, CASH:—						
At Banker's, on Deposit...	150	0	0			
At Banker's, Current Account	36	9	6			
In hand, Subscriptions, &c.	23	3	9			
	209	13	3			
Less Petty Cash	2	18	1			
				206	15	2
,, Balance, being excess of Expenditure over Income, 12 months to date as per account	127	16	6			
Less Surplus as at December 31, 1913...	88	10	5			
				39	6	1
				£384	3	3

We report to the Members that we have obtained all the information and explanations we have required, and that we have examined the above Balance Sheet dated 31st December, 1914, with the Books and Vouchers of the Concrete Institute. We certify that such Balance Sheet is properly drawn up so as to exhibit a true and correct view of the state of the Institute's affairs according to the best of our information and the explanations given us, and as shown by such Books and Accounts.

(*Signed*) MONKHOUSE, STONEHAM & CO.,
Chartered Accountants.
SALISBURY HOUSE,
LONDON WALL, E.C.
April 17, 1915.

FOR THE CONCRETE INSTITUTE:
(*Signed*) HENRY ADAMS, *President.*
H. D. SEARLES-WOOD, *Chairman of Finance Committee.*
H. KEMPTON DYSON, *Secretary.*

Ordinary Members—Professor Henry Adams, Mr. H. K. G. Bamber, Mr. D. B. Butler, Mr. A. C. Davis, Mr. Oscar Faber, Mr. Percival M. Fraser, Mr. Matt Garbutt, Mr. William F. King, Dr. J. S. Owens, Major H. S. Rogers, Mr. A. R. Sage, Mr. E. P. Wells.

THE PARLIAMENTARY STANDING COMMITTEE.

Chairman—Mr. H. P. Boulnois.

Vice-Chairman—Mr. J. Ernest Franck.

Hon. Secretary—Mr. Osborn C. Hills.

Ordinary Members—Professor Henry Adams, Mr. H. H. D. Anderson, Mr. W. E. A. Brown, Mr. J. S. E. de Vesian, Mr. Matt Garbutt, Mr. T. de Courcy Meade, Mr. W. G. Perkins, Mr. Edwin O. Sachs, Mr. L. Serraillier, Mr. E. P. Wells.

The President is *ex-officio* member of all Standing Committees.

In January 1915 Mr. C. S. Meik tendered his resignation as Vice-President and a Member of Council, which was accepted with regret. Mr. Charles F. Marsh was appointed Vice-President in his stead. This will create a vacancy among the ordinary Members of Council.

The Members of Council chosen to retire under the Rules of the Institute were as follows :—

Mr. D. B. Butler	Mr. John Munro
Mr. J. Ernest Franck	Mr. W. G. Perkins
Major R. Napier Harvey	Mr. G. C. Workman

Of these Messrs. Butler, Franck, Perkins, and Workman were eligible for re-election, and their names have been submitted to the Members.

In addition the Council nominated :—

Mr. Ewart S. Andrews	Mr. A. R. Sage
Mr. William F. King	Mr. R. H. Harry Stanger
Mr. Lewis H. Rugg	Mr. John M. Theobald

The Engineering Standards Committee having asked the Institute to nominate a representative to take the place of Mr. W. G. Kirkaldy as representative of the Concrete Institute on the Sectional Committee on Bridges and General Building Construction, the Council appointed Mr. R. H. Harry Stanger in that capacity.

The Council also nominated Mr. H. Kempton Dyson, Secretary of the Concrete Institute, to fill the vacancy created by the death of Mr. Kirkaldy upon the Joint Committee on Reinforced Concrete conducted by the Royal Institute of British Architects.

The Rules and Syllabus of the proposed examination of the Concrete Institute were appended to the previous Report of the Council. The Council has decided that the first examination shall be held early in June 1916, of which due notice will be given.

The Council regrets to record the decease of :—

Hon. Members—

Monsieur Armand Considère, Correspondent de L'Académie des Sciences, Inspecteur Général des Ponts et Chaussées en retraite,
103 Boulevard du Montparnasse,
Paris, France.

Monsieur Edmond Coignet, Ingénieur Civil (E.C.P.),
20 Rue de Londres,
Paris, France.

Member. Killed in Action—

Major A. H. Tyler, R.E., F.R.G.S.,
9 Manor Road,
Salisbury.

Member—

Mr. F. Dare Clapham, F.R.I.B.A.,
Southampton Street,
Bloomsbury, W.C.

Several donations to the Library have been received by the Council from authors, publishers, and kindred societies, and the Council expresses thanks to the donors. A list of books received is published from time to time in the TRANSACTIONS.

FINANCE AND GENERAL PURPOSES COMMITTEE.

The Finance and General Purposes Committee has held regular meetings preliminary to each Council Meeting, and the general results of their deliberations are contained in the foregoing particulars of the Council's work for the year.

Science Standing Committee.

In addition to considering the L.C.C. Regulations for Reinforced Concrete, the Science Standing Committee has been concerned, jointly with the Reinforced Concrete Practice Standing Committee, in the compilation of a Standard Specification for Reinforced Concrete Work which was submitted in draft for discussion at a General Meeting. The Report has yet to be revised in the light of the discussion before it can be re-submitted to the Council for their final approval.

The Science Standing Committee has the following matters under consideration :—

1. Standardization of joints and connections in reinforced concrete.
2. Amendment of the Standard Specification for cement.
3. Co-ordination of the Standard Specification for structural steel of all kinds.
4. The adhesion of and friction between concrete and steel.
5. Reinforced concrete piles.
6. The effect of sewage upon concrete.
7. The effect of oils and fats on concrete.

Reinforced Concrete Practice Standing Committee.

During the past Session the Reinforced Concrete Practice Standing Committee has met, in conjunction with the other Standing Committees and the Council, to consider the L.C.C. Regulations for Reinforced Concrete.

The Committee has held joint meetings with delegates of the Quantity Surveyors' Association and with Members of the Concrete Institute who are Quantity Surveyors by profession. The Joint Committee in the previous Session submitted for discussion at a General Meeting a Draft Report on a Standard Method of Measurement for Reinforced Concrete. During the past Session a number of meetings have been held for the purpose of revising this Report, delegates attending from the Royal Institute of British Archi-

tects, the Surveyors' Institution, the Institute of Builders, and the National Federation of Building Trades Employers of Great Britain and Ireland. As the outcome a final Report upon the Measurement of Reinforced Concrete in Building Construction was submitted to the Councils of the Concrete Institute and the Quantity Surveyors' Association and adopted by them. It has been arranged to continue the work of the Committee with a view to the formulation of a Report on the Measurement of Reinforced Concrete in Engineering Works as distinct from Building Works. At the same time as the Draft Report above referred to was submitted for discussion, another Draft Report of the Reinforced Concrete Practice Standing Committee alone was put forward. The work of revising this report has now to be undertaken. The final report will contain suggestions as to the manner in which engineers should furnish information to quantity surveyors, the nomenclature to be used, and a tabulated form for preparing quantities for reinforced concrete.

The Committee has also been engaged jointly with the Science Standing Committee in the preparation of the Standard Specification for Reinforced Concrete as recorded above.

The Reinforced Concrete Practice Standing Committee has the following matters under consideration :—

1. Advice to clerks of works, inspectors, and foremen as to methods of properly executing concrete and reinforced concrete work and of preventing defects and failures.
2. Regulations, recommendations of joint committees, and various methods of calculation in respect to the design of reinforced concrete and the like.
3. Forms and centering for reinforced concrete work.
4. Standard concrete mixtures for general purposes.
5. The use of cinder, ash, clinker, and breeze in concrete.
6. Means of keeping reinforcements in place when concreting.
7. Methods of making concrete watertight and of waterproofing concrete.

Tests Standing Committee.

The Tests Standing Committee has held joint meetings with the Council and the other Standing Committees, as previously mentioned.

The Tests Standing Committee has the following matters under consideration :—

1. The effect of the presence of sulphur and its compounds in aggregates.
2. The grading of aggregates.
3. The effect on concrete of physical changes : (1) Effect of temperature ; (2) Effect of moisture contents.
4. The effect of the use of sodium silicate on the surface of concrete as affecting reinforcing metal.
5. The erratic results obtained by the Vicat needle in ascertaining the initial setting time of cement.
6. The collection of data regarding the elastic moduli of concrete for stresses within working limits.

Parliamentary Standing Committee.

The Parliamentary Standing Committee has held joint meetings with the Council and the other Standing Committees, as previously mentioned.

It has the following matter under consideration :—

The draft of a Bill promoted by the Society of Architects for the registration of architects.

Investigation Committee.

The Investigation Committee has had under consideration reports of failures on reinforced concrete structures, but as the information contributed was confidential, the results of its deliberations cannot be furnished in the form of a Report.

Joint Committee on Loads on Highway Bridges.

The Joint Committee on Loads on Highway Bridges conducted by the Concrete Institute will shortly meet to consider its final draft Report. It is intended to be presented for discussion at a General Meeting next Session.

APPENDIX

ROLL OF HONOUR.

Names of Members, Associate Members, and Students of the Concrete Institute who are engaged with the Services.

Lance-Corporal C. H. Adams, Divisional Engineers, No. 1 Field Company.

Second-Lieutenant J. G. Ambrose, R.E.

Corporal Peter Baillie, D Company, London Scottish.

Edgar J. Bate, Queen Victoria's Rifles.

Reginald Birkett, Public Schools Battalion of the Royal Fusiliers.

H. Percy Boulnois, The City of London National Guard.

Lieutenant E. Spence Bourne, 5th Batt. Cheshire Regiment (T.F.).

Lieutenant William Campbell Bramham, 9th Leicester Regiment.

Private P. S. Bridson.

Private E. P. Cardell.

Second-Lieutenant A. E. Corbett, 11th (Lonsdale) Batt., The Border Regiment.

Private Cyril P. Crabtree, 7th Welsh Cyclists Batt.

Captain A. T. Cranmer, 8th Middlesex Regiment.

Private Noel Edward Creasy, 5th Batt. East Surrey Regiment (T.F.).

Private P. R. Dunkley, Queen's Westminster Rifles.

Private R. E. Eiloart, R.A.M.C., 3rd London Field Ambulance.

Captain Ernest G. Fowler, North Midland Division A.S.C.

Lieut.-Colonel W. Laurence Gadd, V.D., Kent Royal Garrison Artillery.

Trooper C. F. S. Grove, Machine Gun Section, Royal Westminster Dragoons.

Lieut.-Colonel R. N. Harvey, D.S.O., R.E.

Captain Edgar H. Heathcote, 6th Sherwood Foresters.

Lieutenant H. J. P. Hopkinson, R.E.

Lieutenant J. D. Howkins, R.N.V.R.

Lindow H. L. Huddart, M.A., with the British Forces in the Cameroons.

Lieutenant A. C. Hughes, 4th Royal Berks.

Corporal W. J. Leahy, R.E., 10th Motor Cycle Section, 2nd Corps Headquarters.

Charles Lister, serving in coast defence at Mombasa, British East Africa.

Trooper H. D. North, Middlesex Imperial Yeomanry.

Captain H. P. Oldfield, 3rd Wiltshire Regiment.

Private C. J. Pell.

Private Mark Pitter.

Second-Lieutenant Gerald H. Reed, Cornwall Fortress R.E. (Works Company).

Lieutenant L. H. Rugg.

Lieutenant Reginald Ryves, R.E. (T.F.).

Second-Lieutenant Walter Saise, R.E.

Major W. H. Stanbury, Staff for Royal Engineer Services.

Private R. Stubbs, R.A.M.C.

Private John M. Theobald, National Reserve.

Private Philip H. Thorne.

Second-Lieutenant Cecil Upcher, 9th Norfolk Regiment.

Second-Lieutenant T. H. Upton.

Second-Lieutenant G. T. Uren, 6th Worcestershire Regiment.

Lieut.-Colonel Thomas Thornycroft Vernon, 7th Reserve Batt. The King's (Liverpool Regiment).

Private B. B. Willcox.

Private H. W. Williams, King's Liverpool Rifles (Scottish).

DISCUSSION

THE PRESIDENT :—It is now my duty to put before you the annual report and balance-sheet, and as we have some considerable business this evening following the Annual General Meeting I will, with your permission, take it as read. You have copies before you, and I would just call your attention to a few of the more striking points in it. On the first page you will observe, that in spite of all difficul-

ties the membership has not gone down, but has still continued to advance during the last three years. It is a matter for congratulation, that there has not been a considerable decrease in membership owing to the war, although a number of members of the Institute are serving with the Forces. As regards the work of the Council and Committees, you will be glad to hear, that the Reinforced Concrete Regulations of the London County Council have at last passed through all their stages of revision, which have occupied some years, and that they are now in the hands of the Local Government Board and London County Council for presentation to the Council and ultimately to the public. We are very well satisfied with the result, and I think that when you see the document you also will be satisfied. It does not profess to be perfect, but it is as good as the combined efforts of all the various institutions concerned could make it, and the County Council has met us very fairly and in a friendly spirit with regard to our suggestions. With regard to the balance-sheet, which is the next thing we come to, and the income and expenditure account, you will notice that it says the balance in excess of expenditure is £127 16s. 6d. I would point out, that we have put by a sum of £150 towards our printing account for next session, because the printing has been somewhat delayed owing to the war. I propose the adoption of this report and balance-sheet.

MR. MORGAN E. YEATMAN, M.A., M.Inst.C.E., M.Am.Soc.C.E., M.C.I. :—I have much pleasure in seconding the motion for the adoption of the report and balance-sheet, and I do not think I need add anything to what our President has said.

THE PRESIDENT :—I will put it to the meeting that the report of the Council and the balance-sheet be adopted.

Agreed.

THE PRESIDENT :—I will now call upon the Secretary to read the report of the Scrutineers in the Annual Election for the Members of the Council.

THE SECRETARY :—We, the Scrutineers to the Council Election of the Concrete Institute, have found as follows :—

Stanger, R. H. H.	...	135
Butler, D. B. ...	...	127
Owens, J. S. ...	...	120
Perkins, W. G. ...	...	120
Andrews, E. S. ...	...	119
Franck, J. E. ...	...	116
Rugg, L. H. ...	...	107
Workman, G. C....	...	99
De Vesian, J. S. E.	...	84
Sage, A. R. ...	...	78
Theobald, J. M....	...	67
King, W. F. ...	...	61

One hundred and fifty-nine voting papers were received. Nine lists of attendances were returned instead of voting papers. The votes of seven persons had to be ignored owing to their subscription not having been paid. One paper was rejected owing to the envelope not having been signed.

Yours truly,

(Signed) R. H. LACY.
S. BYLANDER.

THE PRESIDENT :—The last three names on that list are not elected. The others are elected by reason of the majority of the voting.

MR. ALEX. C. MESTON, Licentiate R.I.B.A., M.C.I. :—I beg to propose that Messrs. Monkhouse, Stoneham & Co. be elected the Auditors of the accounts for the current year at a fee of five guineas.

MR. PERCY H. SIMCO, M.C.I. :—I will second that.

Agreed.

THE PRESIDENT :—I have now a very pleasing duty to perform. It is the custom to present the medal of the Concrete Institute to the author of the best paper read during the year at the Ordinary Meetings of the members. Where so many good papers are received it is a matter of considerable

difficulty to select one, that is pre-eminent, but in the present instance I think you will agree, that the selected paper was an uncommonly good one. Of course, where there are members practising in different branches, different papers will appeal to them in a different manner. This paper is the one that received the highest number of the votes of the Council, which gives the medal. I may tell you, that it was a postal vote, so that all members of the Council had an equal opportunity of recording their opinion. The result was, that the paper by Mr. Cyril Cocking, entitled "Steel-frame Buildings from a Draughtsman's Point of View," was adjudged to be the best, and I have very much pleasure in asking his acceptance of the Institute medal as a record of the valuable work, he has done to further the interests of the Institute. I have much pleasure in presenting this medal to you, Mr. Cocking, with the best wishes of the Council.

Mr. W. Cyril Cocking, M.C.I., in returning thanks for the Concrete Institute bronze medal, said it had been suggested that some personal remarks anent the structural engineering profession generally would be fitting upon this occasion. The question for us as an engineering institution and an educational institution is, whether we shall be ready, when the war is over and the period of recuperation and reconstruction is upon us, to take and hold the foremost position among the civilized nations of the earth. Shall we be in a position to send out competent engineers and competent draughtsmen to repair the damage, the war has caused, and shall we be able to capture Germany's engineering trade, in addition and without prejudice to the building and engineering work, which we always have on hand in this country? This is a problem of supreme importance to us. It looks as if there will be a further war for commercial and professional efficiency, and efficiency in the engineering profession is almost, if not entirely, a question of education. It is a matter of national importance, that the education of the structural and constructional engineer should be placed upon a sound and up-to-date basis. The classes of instruction

should be so arranged as to give the necessary continuity of purpose, in order that the deplorable wastage of students' time, which now obtains, shall be avoided. It is also necessary, that the instruction given should be uniformly and well prepared by the instructor and easy of assimilation and retention by the student. In this connection I would suggest, that steps be immediately taken by this Institute to inquire as to ways and means whereby the educational system of this country, particularly with respect to structural engineering, can be improved and extended, in order to ensure that the British structural engineer shall hold the supremacy in every quarter of the globe. There should also be some means provided, so that the best advice can be given to the student leaving the day-school to enable him to fit himself as a structural engineer in the shortest possible time. I am convinced, that if this Institute were to consider the educational question as directly applied to our profession, and draw up a comprehensive and well-conceived scheme, the educational authorities would come up to the mark and see the thing through.

The second question I would bring before your notice is the necessity for some qualification for the structural engineer. It seems to me a curious anomaly, that a structural draughtsman with a deficient theoretical training and incomplete practical experience can be, and often is, allowed to design, without supervision, structures upon the safety of which the lives of his fellow-men depend, whereas in the medical profession a rigid training and a certificate of competency are required before the medico can commence to practise at all. In the past I have designed the steelwork for theatres and other places of public assembly with only the most meagre supervision, at a time when I could not consider myself fully competent, and yet the lives of hundreds of people have been, and are still, entirely dependent upon the accuracy of my calculations, theories, and details. Surely the structural engineer should be required to show some certificate of competency before he is allowed to design without supervision. On page 3 of Mr. Hills' paper he suggests that, from evidence which he gives, the only inference we can draw is,

that the erection of buildings should only be undertaken by properly qualified individuals. With regard to this statement I am persuaded, that we are all of one mind, but are we to wait until some appalling disaster takes place in our midst before the importance of a proper qualification is brought home to the "man in the street" and the responsible engineer? The question of responsibility is often confused. Some people will argue that, because a public building must be designed to the satisfaction of a District surveyor, he (the District surveyor) takes full responsibility, or if the design is supervised by a consulting engineer, the latter takes the responsibility. I personally cannot accept that view. To my mind the responsibility rests with the designer, whether his work is supervised and checked, or not. The designer, who refuses responsibility for his own designs, declares himself incompetent and unqualified by so doing. The question is, "What should be the qualification of a structural engineer?" That is for this Institute to decide. There should be a theoretical qualification and a practical qualification, but the one is useless without the other. Why should not the Concrete Institute issue certificates of competency to competent men? They would be of inestimable value both to employer and employee, and incidentally to the District surveyor. My own personal view—and I would accentuate the personal—is that it would have been better for the engineering profession, if the 1909 Act had never been promoted, but that an Act of Parliament had been passed compelling structural engineers to register themselves, and empowering the Concrete Institute, together with the other institutions directly interested, to formulate rules to be observed both in the design and the carrying out of reinforced concrete and steel structures. That is constituting our branch of the engineering profession on a basis similar to the legal profession and the Law Society. It is difficult to draw a comparison, but an Act like the 1909 Act does not encourage the professional man. It only encourages the incompetent, by giving him rules to work to, of which he does not appreciate the value and importance. In fact, the whole aim, especially of the Reinforced Concrete Regulations, is to make a

Building Act what you might call fool-proof! The Act has certainly improved design, but only up to a certain point. It does not allow for future improvement and economy, and it destroys originality. I personally would much prefer, that the engineer should be properly qualified and then be allowed a free hand within reasonable limits.

MR. HENRY J. TINGLE, M.Inst.C.E., M.C.I., proposed a vote of thanks to the President, which was seconded by MR. S. BYLANDER (Past Chairman J.I.E.), M.C.I.

The vote was carried with acclamation.

THE PRESIDENT briefly responded.

SIXTY-FIRST ORDINARY GENERAL MEETING

THURSDAY, APRIL 22, 1915

THE SIXTY-FIRST ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, April 22, 1915, at 7.30 p.m.,

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

MR. FREDERICK BROADHURST HAMILTON, A.M.I.Mech.E., Departmental Manager and Chief Engineer of Messrs. British Coke Ovens, Ltd., 7 Princes Street, Westminster, was elected a member ; and the admission to studentship of MR. THOMAS ERNEST DODDS, Hons.B.Sc.Eng.(Lond.), Whitworth Scholar, Assistant Engineer to Messrs. Henry Adams & Son, 60 Queen Victoria Street, London, E.C., was announced.

MR. F. E. WENTWORTH-SHEILDS, M.Inst.C.E. (Vice-President C.I.), read his paper, entitled :—

THE STABILITY OF QUAY WALLS ON EARTH FOUNDATIONS

IN spite of the large amount of experience which has been gained in the construction of quay walls, it is still one of the most difficult problems in engineering to design a wall on an earth foundation with confidence that it will be stable when completed. A warehouse or a bridge can now be designed, not only with the assurance that it will bear its load, but also with a knowledge of its factor of safety sufficiently accurate to satisfy the designer that material has not been wasted. But the same can certainly not be said of a retaining wall on a soft bottom—at all events not of a wall, say, 40 to 80 ft. high, such as is commonly needed to sustain the quays of a modern dock. Even if the designer of such a wall is assured that it will stand, he cannot with any confidence tell you what factor of safety it possesses. The cause of his

uncertainty is of course the difficulty of ascertaining the actual lateral pressure imposed by an earth backing and the actual resistance offered by an earth foundation. His difficulties are thus quite different from those of the engineer who has to design large masonry dams. The latter structures are invariably placed on a foundation of solid rock, and the designer's chief care is that the stresses in the masonry of which the dam is composed shall not exceed a safe limit. The dock engineer on the other hand has to be anxious that his wall shall not move as a whole on the comparatively soft material on which such structures have in general to be placed.

The object of this paper is to consider the uncertainties and difficulties which the designer of a quay wall has to face, and, if possible, to ascertain how far calculations can assist him, and how far he must trust to judgment based on experience. It is also intended to urge upon this Institute the importance of collecting information on this subject in the hopes that by degrees these difficulties and uncertainties may be cleared away.

A retaining wall may fail as a whole in two ways—

(1) By sliding forward on its base, and
(2) By overturning.

It may be said at once that as regards quay walls, at least, the former mode of failure (by sliding forward) is by far the more common.

The conditions of stability in a "gravity" wall may be summarized thus—

The forces tending to thrust the wall outwards (generally the lateral pressure of the earth backing) must be at least equalled by the forces tending to restrain it or thrust it inwards. The latter forces are generally the pressure of the water in front of it, the resistance of the earth in front of its toe, and the horizontal resistance to shear (or the friction) at the base of the wall. If these horizontal forces balance, the wall cannot slide forward.

The resultant of the outward forces, however, is almost always at a higher level than the resultant of the inward forces. Thus a couple is formed tending to overturn the wall about its toe. This couple induces a counter-couple tending to keep it upright. The forces forming this counter-couple consist, on the one hand of the weight of the wall acting vertically downwards, together with the weight of any earth or water which may lie above the base of the wall, and on

the other hand the upward resistance of the earth under that base.

If the upward resistance of the earth beneath the wall is capable of forming with the downward weights a couple at least equal to the overturning couple, the wall cannot overturn.

In order that the earth beneath the wall shall be capable of forming this righting couple, two things are necessary. It is obvious that the centre of the earth's resistance must be forward of the centre of gravity of the wall and of other loads on the base, and generally it is forward also of the mid-point of the base of the wall. Consequently the intensity of upward resistance is generally greatest at the toe and least at the heel. To preserve stability the resistance at the toe must not be greater than the maximum which the earth is capable of offering, and that at the heel must not be less than the pressure induced by the tendency of the earth to rise at this point.

Thus it will be seen that if the stability of any given wall is to be ascertained by calculation, the following forces must be determined :—

(1) The amount and position of resultants of horizontal outward forces such as—

Lateral pressure of backing.
Lateral pressure due to surcharge.
Pull of ships' moorings on bollards.

(2) The amount and position of resultants of horizontal inward forces such as—

Pressure of water in front of wall.
Resistance of earth in front of toe.
Resistance due to friction at base of wall.

Note.—The forces of paragraph (2) must be at least equal to those of paragraph (1).

(3) The overturning moment produced by inward and outward forces.

(4) The amount and position of resultants of vertical downward forces such as—

Weight of wall.
Weight of backing resting on wall.
Weight of water on toe.
Friction between backing and back of wall.
Friction between toe and earth in front of it.

(5) Consequent amount and position of resultant of upward vertical forces such as—

Reaction from earth under base.

Note.—The position of this resultant can be obtained by equating the righting moment produced by the vertical forces to the overturning moment due to the horizontal forces.

(6) The consequent maximum and minimum intensities of reacting pressure of earth under base at toe and at heel respectively.

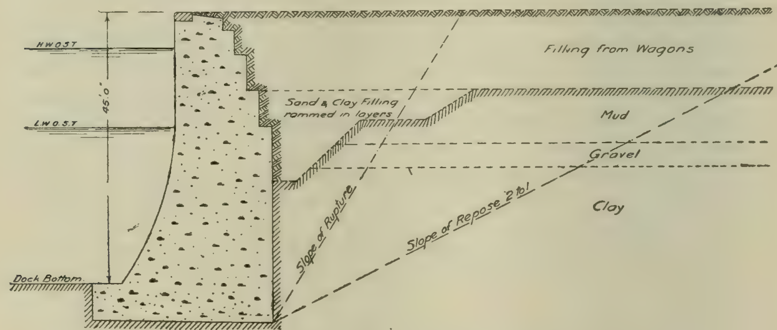


FIG. 1.

(7) The maximum permissible reacting pressure at toe, and the minimum permissible pressure at heel.

Note.—The actual pressure at the toe (referred to in paragraph (6)) must not be greater than the permissible pressure (referred to in paragraph (7)), and the actual pressure at the heel must not be less than the permissible pressure.

This method of ascertaining the stability of a wall is illustrated by an actual case in Appendix I. The same results can of course be obtained by graphic methods.

The case in question is that of the quay walls of the Empress Dock at Southampton. Before considering the results of the calculations given in the appendixes, it will be well to state shortly the conditions under which these walls were built, and their behaviour under these conditions.

The Empress Dock is a four-sided tidal basin, each side being about 800 ft. long. During construction the tide was excluded by means of an enclosing bank. A section of the quay wall is shown in Fig. 1. It is made of concrete and

is 30 ft. wide at base, 45 ft. high from cope to dock bottom, and 51½ ft. high from cope to foundation at back. It rests on a moderately firm sandy clay. The lower part of the wall is backed by the same clay in its virgin state, as it was built in a timbered trench. The upper part is backed with much the same material excavated from the dock and cast out and rammed in layers. The angle of repose of the clay was about 26°. After the backing had been raised to cope level and the ground in front of the wall had been removed, the north wall of the dock moved forward. The maximum movement was about 23 ft., but the wall maintained its upright position, showing that the movement was not due to

FIG. 2.

any overturning effect, or to crushing of the clay underneath its toe, but simply to the fact that the resistance of the clay in front of the toe plus the friction at the base was insufficient to balance the lateral pressure of the backing.

This wall was taken down and rebuilt to the same section, except that the foundations were carried down to 15 ft. below dock bottom instead of 6½ ft., and the result has been quite successful (see Fig. 5).

At the same time in order to try and prevent a similar accident to the other walls, large blocks or buttresses were built in front of the toe. These buttresses were each 20 ft. long, 15 ft. wide, and 12 ft. deep, the top of each being level with the dock bottom, and the base 12 ft. below. They were spaced 30 ft. apart in the clear. It was thought that the buttresses, being 6 ft. deeper than the wall, could not plough up the ground in front of them, and would therefore resist its lateral movement. This hope was vain, however.

Another of the walls (the east wall) moved forward, buttresses and all, in the fashion shown in Fig. 2. It will be noticed that so far from overturning about its toe, the wall overturned backward about its heel.

In order to save the west wall after this experience, besides building the buttresses in front of the toe, the backing was removed to a depth of about 13 ft. below quay level (see Fig. 3), and by this means serious movement was prevented until the water was admitted to the dock. The backing was not restored, the quay being carried on a viaduct, as shown in Fig. 3. Even under these circumstances the quay was constantly showing slight signs of movement, and eventually elaborate strengthening works had to be undertaken, which need not be described here.

It will be interesting now to examine how far the movements of this wall could have been foretold, or can be accounted for by calculating the forces at work, and thus to form some idea of how far such calculations may be relied upon.

The calculations given in Appendix I relate to the wall as originally built, fully backed, with the earth in front removed to dock bottom, and with no tidal water in the dock. The calculations are based on Rankine's formulæ, and assume a value of 26° for ϕ, the angle of repose, and 0·3 for the coefficient of friction at the base of the wall, and give the following results :—

Outward horizontal forces—		
		Tons.
Pressure of backing		27·8
Inward horizontal forces—		
	Tons.	
Resistance of earth at toe	2·9	
Resistance due to friction at base	18·6	
		21·5
Showing an excess of force tending to move the wall of		6·3

As a matter of fact the wall stood for a week under these conditions, so that during this time the excess of outward force of 6·3 tons cannot have existed. Possibly the friction at base was at first greater. Assuming this to be the case, the overturning moment is found to be 468 ft.-tons. The total vertical loads on the base of the wall are found to be 62 tons, and by equating the righting moment due to same with the overturning moment it is found that the centre of

reaction from the base is 2·8 ft. from its middle point. Hence the pressure at toe is found to be 3·2 tons per square foot, and the minimum pressure at heel 0·9 ton per square foot. The maximum permissible pressure at toe is found to be 2·3 tons per square foot, and the minimum permissible pressure at heel 0·4 ton per square foot.

It will be seen that these calculations suggest that the wall is unsafe as regards the tendency both to slide forward, and to overturn by crushing the earth under its toe. As a matter of fact, it failed by sliding forward but showed no sign of overturning.

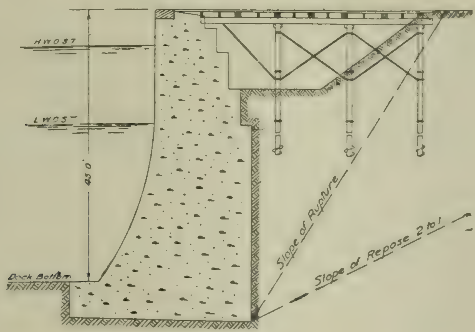


FIG. 3.

We will now pass on to consider the case of the west wall, under the conditions shown in Fig. 3, i.e. with a portion of the backing removed and with the tidal water admitted into the dock. Adopting the same values for the angle of repose of the backing, 26°, and for the coefficient of friction at the base, 0·3, the balance of horizontal forces is as follows (see Appendix II) :—

Outward horizontal forces—		
		Tons.
Pressure of backing		22·0
Inward horizontal forces—		
	Tons.	
Pressure of water	9·7	
Resistance of earth at toe	2·9	
Resistance due to friction at base...	21·0	
		33·6
Showing an excess of force tending to keep the wall stable of		11·6

These calculations, unlike the last, suggest that the wall is quite safe as regards the tendency to slide forward. But as a matter of fact it *did* slide forward slightly, and was certainly not as safe as these calculations suggest. Possibly the coefficient of friction at the base, which at one time must have been more than 0·3, had now got reduced by the lubricating action of water which had crept in under the wall to about half that figure, or else the lateral pressure of the backing had become increased, owing to the softening action of the water. It is probable that the former explanation is the right one. At all events the history of this wall shows how difficult it is to make exact calculations for the stability of large quay walls, and that the values to be given to such constants as ϕ and f in the formulæ used, must be largely a matter of individual judgment based on experience.

It follows that calculations for the stability of quay walls cannot in our present state of knowledge be always relied upon, and that failures on the one hand, and waste of material on the other, are liable to occur even to the most careful and experienced designer. This, however, is not saying that such calculations should be ignored. On the contrary they are most useful in suggesting means for increasing in the most economical manner the stability of designs, which are known or suspected to be weak.

We will therefore consider a little further some of the calculations which have been referred to in this paper.

First as regards the lateral pressure of the earth backing, Rankine's formula is perhaps most commonly used. An objection often raised against it is that the resultant pressure given is too high, as it neglects the friction between the backing and the back of the wall. A modification of it by Bousinesq, which takes this friction into consideration, is sometimes adopted. It is quite conceivable, however, that a wall may commence to slide on its base without calling into play the friction between wall and backing, so that it can be neglected without error so far as the balance of horizontal forces on the wall is concerned. On the other hand, it probably appreciably helps to resist the tendency of the wall to overturn—a matter which will be considered later. A much more important fact about Rankine's formula is that it is really only applicable to granular materials like sand, and not to plastic materials like clay. This has always been more or less recognized, and has been attributed to the fact that the angle of repose of a given clay is liable to change according to whether the clay is in its virgin state, or whether

it has been exposed to weather or other adverse conditions. But it has been shown by Mr. A. L. Bell, B.A., in a paper recently read by him before the Institution of Civil Engineers, that Rankine's formula is quite inapplicable to clay, even in a given state, and that its use in the case of high walls may give very much lower values for the horizontal pressure of clay backing than really obtain. As the result of his investigations he proposes the following formula for the intensity of pressure at any point behind the wall :—

$$p = w\,h \tan^2\left(\frac{\pi}{4} - \frac{a}{2}\right) - 2\,k \tan\left(\frac{\pi}{4} - \frac{a}{2}\right)$$

where p = intensity of horizontal pressure (tons per square foot),
w = weight of backing (tons per cubic foot),
h = height from point considered to surface,
a = a constant (angle),
k = a constant (tons per square foot).

The values of k and a can be determined for any given clay by experiment with a very ingenious apparatus devised by Mr. Bell, who gives the following approximate values for these symbols :—

	k Tons per sq. ft.	a Degrees.
Very soft puddle clay... ...	0·2	0
Soft puddle clay	0·3	3
Moderately firm clay	0·5	5
Stiff clay	0·7	7
Very stiff boulder clay ...	1·6	16

It will be seen that according to this formula, the pressure down to a certain depth below the surface is nil, but below that depth it increases rapidly—very much more rapidly than Rankine's formula would suggest. This is tantamount to saying that Rankine's formula overestimates the pressure on low walls backed with clay, and underestimates the pressure on high walls. This accords generally with experience. In the case of granular materials, such as sand, k becomes nil, and a becomes ϕ the angle of repose (or nearly so), and the formula then becomes identical with Rankine's, since—

$$\tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) = \frac{1 - \sin\phi}{1 + \sin\phi}$$

Appendix III gives the horizontal pressure on the Empress Dock Wall, calculated by Bell's formula under the same conditions of loading as were assumed in Appendix I, except that the clay down to $28\frac{1}{2}$ ft. below cope is assumed to be "soft puddle clay," and below that "moderately firm clay." The resultant pressure (27 tons) is, as it happens, much the same as given in Appendix I by Rankine's formula assuming $\phi = 26°$. It will be seen later, however, that in the case of a higher wall than the one here considered, the value of P given by Bell's formula would be greater than that given by Rankine's. On the other hand, with a reduced depth of backing as considered in Appendix II, Bell's formula would give a lower value for the value of P than Rankine's. In a case like this, where the backing consisted of sandy clay in layers interleaved with thin beds of sand, it is most difficult to say which formula gives the more accurate results.

In the case of gravel or sand behind a quay wall, the backing is always charged with water, and the question arises, does this water increase the lateral pressure? The author has endeavoured to determine this by experiments, but so far without any satisfactory results. He has generally assumed in his calculations that in the case of sand and sandy gravel the water has no effect, except that of increasing the weight per cubic foot of the backing, but that in open materials like broken stone, the lateral pressure is similar to that produced by water alone.

The curves on Figs. 12 and 13 enable the results of Rankine's formula to be obtained without calculation. Knowing the weight per cubic foot and the angle of repose of any material, Fig. 12 gives the weight (or pressure) per cubic foot of the equivalent fluid, and Fig. 13 the corresponding total pressure per foot-run on any wall of known height.

Fig. 14 shows a series of curves from which can be seen the intensity of horizontal pressure of various kinds of clay backing at any given depth below the surface, the results being in this case obtained from Bell's formula.

Several other formulæ for the lateral pressure of earth have been devised, but enough has been said to show that Rankine's and Bell's formulæ probably give sufficiently accurate results in the case of high quay walls, if only the values of the constants can be accurately determined, and that the prevailing idea that Rankine's formula gives results which are much too high, is not altogether correct.

There are, of course, forces on a quay wall tending to move it outwards other than the pressure of the backing, such as

the pressure due to surcharge on the quay, and the pull of vessels on the bollards. These should be considered, though it will be found that in high walls they are small in comparison with the pressure of the backing. Thus in the White Star Dock Wall, shown in Fig. 7, the bollards are designed for a safe pull of 50 tons. Even if this pull were normal to the line of wall, the horizontal reaction at the base of the wall would probably be less than 1 ton per foot-run, as against some 57 tons per foot-run due to the pressure of the backing.

Coming to the "inward" forces, or those tending to resist outward movement, the pressure of the water in front of the wall is always taken into consideration, and rightly so, except in cases where the dock is liable to be dried out. In a tidal dock the water level is taken to be that of the lowest tide.

The resistance due to the earth in front of the toe is generally calculated by Rankine's formula—

$$r = w\,d\,\frac{1+\sin\phi}{1-\sin\phi}$$

where r = intensity of resistance (tons per square foot) at any depth d (feet),

w = weight of earth (tons per cubic foot),

ϕ = angle of repose,

or the total resistance per foot-run of wall (tons)—

$$R = \frac{w\,d^2}{2}\,\frac{1+\sin\phi}{1-\sin\phi}$$

For the first of these formulæ Mr. Bell proposes to substitute in the case of clay—

$$r = w\,d\tan^2\left(\frac{\pi}{4}+\frac{a}{2}\right) + 2\,k\tan\left(\frac{\pi}{4}+\frac{a}{2}\right)$$

where a and k have the same values as given in his formula for lateral pressure of clay backing (see page 181).

It is interesting to note that if this formula be applied to the Empress Dock Wall, the foundation being regarded as a "moderately firm clay," it gives a value for R of 8·4 tons (see Appendix III), as against 2·9 tons by the Rankine formula. In view of the movement that took place in this wall, even after

the buttresses had been built in front of it, and the tidal water admitted to stabilize it, it is very doubtful whether the resistance had such a high value as 8·4 tons. As mentioned previously, the clay in this case was in layers interleaved with thin beds of sand, and in such a case it is probable that Bell's formula would give too optimistic results for the horizontal resistance of the material.

It is a question whether the weight of any water above the earth in front of the toe should be considered as adding to its stability. Probably it is right to neglect this, as if the water can penetrate into the earth, its weight is counteracted by its lifting effect and nothing is gained.

In most quay walls the greatest "inward" force is the resistance due to friction at the base of the wall. At the same time its amount is most difficult to determine on account of the uncertainty of the coefficient of friction there. Its value as between concrete and various earths is sometimes given as follows :—

Dry gravel or sand or clay	0·5 to 0·6
Moist " " "	0·3
Wet clay	0·1 to 0·2

These values are probably not far wrong, but in the case of clay they offer an embarrassing choice of values varying from 0·1 to 0·6, according to the state of the clay, which, after all, in the case of a quay wall can only be conjectured.

It is evident that the wall in sliding forward may either glide over the surface of the earth or may shear the earth itself. The resistance to the latter could presumably be obtained in the case of granular material from the formula—

$$R = W \tan \phi$$

A corresponding formula for clay might be deduced from Mr. Bell's investigations, and would be—

$$R = B\,k + W \tan a$$

where R = total resistance to shear per foot-run of wall (tons),
B = width of base of wall under load (feet),
W = total weight on base (tons),
$\left.\begin{matrix} k \\ a \end{matrix}\right\}$ = constants as above.

It will be of interest to apply these two formulæ to the case of the Empress Dock Wall examined in Appendix I.

The first gives a value for R of $62 \times 0{\cdot}48 = 30$ tons, assuming $\phi = 26°$, and the second a value for R of $(30 \times 0{\cdot}5) + (62 \times 0{\cdot}09) = 21$ tons, assuming $k = 0{\cdot}5$ and $a = 5°$.

It is likely, however, that a wall with a horizontal base does not shear the material beneath it, but slides over that material. It will have been noticed by those who have had to underpin concrete walls that a soapy surface is formed underneath the base by the laitence which exudes from the concrete. This surface has undoubtedly a low coefficient of friction. For this reason the author generally assumes that the coefficient of friction at the base of a quay wall may be as low as 0·2 on clay or sandy clay which is fairly firm, and 0·4 on clean sand or gravel. Possibly this assumption errs on the safe side. On the other hand, as we have seen, if water can penetrate under the wall, the coefficient of friction may become even less than this.

Having obtained the value of the outward and inward horizontal forces, the question arises by how much should the latter exceed the former. A factor of safety of 3 has been recommended, but it is very doubtful whether this is ever obtained in large quay walls on clay. If the dock engineer were really assured that he had a factor of 2, he might be well content.

Having obtained the amounts and positions of all these horizontal forces acting on the wall, it is a simple matter to ascertain the overturning moment produced by them. This, of course, can be done by calculation as suggested in Appendix I, or graphically.

We now come to consider the vertical forces on the wall in order to obtain the maximum and minimum reacting pressures from the foundation at the toe and heel respectively.

It is customary to include among these vertical forces besides the weight of the wall itself, the weight of any backing which may lie above the base, and any water which rests on the toe.

It is common, however, to neglect the friction between the backing and the wall, and also between the toe of the wall and the earth in front of it. This procedure is, of course, a safe one, but adds to the uncertainty about the factor of safety. As has already been mentioned, it is very rarely that one finds a quay wall overturning or crushing the earth under its toe, though there are many walls in existence which "on paper" ought to do so. This is probably largely due to the righting moment which is really induced by the vertical friction of the backing, but which has been neglected in the

calculations. Appendix IV shows the effect of considering this vertical friction between the earth backing and the wall in the case of the Empress Dock Wall.

In Appendix I it is shown that the horizontal pressure of the backing is about 27·8 tons. This force might well produce a vertical friction tending to prevent overturning of, say, $27{\cdot}8 \times 0{\cdot}3 = 8{\cdot}3$ tons. Neglecting this force the vertical pressure of the toe is found to be 3·2 tons per square foot, and at the heel 0·9 ton per square foot. But if this vertical 8·3 tons had been considered, the vertical pressure at toe and heel would be modified to 2·7 and 2·0 tons per square foot respectively—or in other words, the pressure becomes nearly uniform throughout the base. This corresponds much more nearly with the actual behaviour of the wall.

One is inclined therefore to suggest that this friction be taken into consideration when calculating the vertical forces which tend to prevent the wall overturning, but neglected when considering the horizontal forces which tend to thrust the wall forward.

And now, when the designer is satisfied that he has obtained something like a true value for the vertical pressure at toe and heel, he is confronted with another difficulty when he has to decide what is the permissible pressure at these points, and what factor of safety he possesses.

Several tables of maximum safe permissible pressures have been published, of which the following is a sample :—

On hard clay	2 to 3	tons per square foot.
On sand and gravel	1½ to 2	,, ,, ,, ,,

Probably these figures are quite safe, so safe that they are never attained to in high quay walls.

Rankine's formula for the greatest permissible pressure on granular materials is—

$$r = w\,d + w_{,}\,d_{,}\left(\frac{1 + \sin\phi}{1 - \sin\phi}\right)^2$$

where the symbols have the significance given in Appendix I. This formula is often condemned, or at least ignored on the ground that it involves the consideration that the supporting power of the material varies directly as its depth below the surface, and is nil at the surface. In the case of

really granular materials, however, like clean sand and gravel, this consideration probably accords with the facts, although further experiments are needed to verify it. In the case of clay it is undoubtedly wrong, and the formula suggested by Mr. Bell is probably much more correct.

It runs—

$$r = (w\,d + w_{,}\,d_{,}) \tan^4\left(\frac{\pi}{2} + \frac{a}{2}\right) + 2\,k \tan^3\left(\frac{\pi}{4} + \frac{a}{2}\right)$$
$$+ 2k \tan\left(\frac{\pi}{4} + \frac{a}{2}\right)$$

where the symbols have the same significance as given in Appendix III.

It is interesting to note that for the case of the Empress Dock Wall, assuming the same values for the symbols as before, the maximum possible pressure under the toe of the wall would be by Rankine's formula 2·3 tons per square foot and by Bell's formula 2·9 tons per square foot. The actual pressure exerted by the wall was, as we have seen, something like 2·7 tons per square foot.

Now as it happens an experiment was made on the sandy clay on which this wall was built, from which it was found that it could support a pressure of $3\frac{1}{4}$ tons per square foot on the surface. Probably at a depth of 6 ft. below the surface, the supporting power would be about $3\frac{3}{4}$ tons per square foot, but even so the factor of safety would be under 2. As high quay walls go, however, this wall with its wide toe may be regarded as an unusually safe one in this respect. A factor of safety of 2 might therefore be considered quite satisfactory for general use.

If Rankine's or Bell's formulæ are used, it would seem rational to consider as part of the weight on the earth in front of the toe the overlying water, as this would undoubtedly assist to keep the earth from rising.

The last calculation which we have to consider is the intensity of vertical reacting pressure at the heel of the wall. This should not exceed the tendency of the earth underneath the heel to rise. This latter pressure is generally found from the formula—

$$r - w\,h\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2$$

Mr. Bell has devised a modification of this formula for use with clay, which runs as follows :—

$$r = wh \tan^4\left(\frac{\pi}{4} - \frac{a}{2}\right) - 2k \tan^3\left(\frac{\pi}{4} - \frac{a}{2}\right) - 2k \tan \left(\frac{\pi}{4} - \frac{a}{2}\right)$$

where the symbols have the same value as given on page 181.

Whichever formula is used the value of r (the intensity of upward pressure at heel) is usually a very small one ; and generally it may be said that provided the resultant of upward reacting forces is well within the middle third of the base, there is nothing to fear from the rising tendency of the material under the heel of the wall.

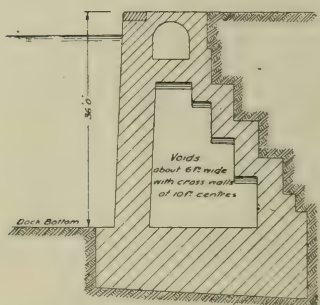


FIG. 4.

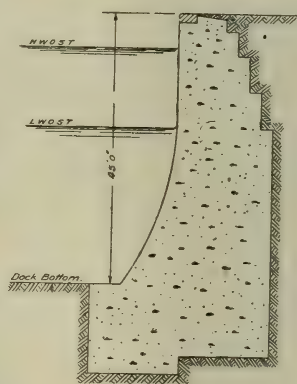


FIG. 5.

We will now consider some devices which have been used for increasing the stability of quay walls and inquire how far calculation and experience would lead us to depend upon them.

(1) *Admission of water into dock.*—Most quay walls are designed to have a certain depth of water in front of them, although in constructing such walls the water is often temporarily excluded, so as to cheapen construction. Calculations would indicate that the pressure due to this water is a great stabilizer, and experience undoubtedly bears this out. A notable instance is the case of the dock wall at Calcutta, described in papers by Messrs. Bruce and Apjohn in Vol. cxxi of the *Minutes of Proceedings Inst. C.E.* The wall is shown on Fig. 4, and is 46 ft. high, and founded on a

material which was said to resemble putty. In spite of the fact that the earth was left in front of the wall for a depth of 26 ft. above foundation, the wall began to move forward, when the backing was raised to within 2½ ft. of cope level. The movement was arrested, however, by admitting water into the dock to a level of 42 ft. above foundation, and no further movement took place, even when 16 ft. more earth was dredged out from in front of the wall. Of course, if the earth had all been removed before water was admitted as was done at Southampton, the wall would undoubtedly have collapsed.

It follows that during construction in the dry, a certain amount of earth should always be left in front of a quay wall until the water is admitted, unless the wall is designed to be stable without any water in front of it.

(2) *Sinking foundations deeper.*—Calculations would indicate that the forces tending to resist sliding forward can be considerably increased by sinking the foundations deeper, and experience confirms this conclusion. For instance, as mentioned above, the Empress Dock Wall which failed was successfully replaced by one of similar design, but carried down to 15 ft. below dock bottom instead of 6 ft. (see Fig. 5). The calculations on the deepened wall show that although the outward pressure on the earth backing was increased from 27 tons to 37·6 tons, the resistance of earth in front of toe was increased from 2·9 tons to 15·4 tons, and the friction at the base from 18·6 tons to 23·4 tons; consequently the balance of horizontal forces was as follows :—

Outward horizontal forces—		
		Tons.
Pressure of backing		37·6
Inward horizontal forces—		
	Tons.	
Resistance of earth at toe	15·4	
Resistance due to friction at base...	23·4	
		38·8
Showing an excess of force tending to keep the wall stable of		1·2

It is interesting to note that much the same balance on the safe side is obtained if Bell's formulæ are used, but the individual forces are greater.

(3) *Building buttress walls in front of main wall.*—Let us

now consider the device which was adopted in order to try to prevent movement on the other walls of the Empress Dock, after the north wall had failed. As previously described, buttresses of concrete 20 ft. × 15 ft. × 12 ft. deep were constructed in trenches in front of the wall toe, at intervals of 30 ft. It was thought, and calculations would seem to suggest, that the resistance to horizontal movement would be increased, as the buttresses were 6 ft. deeper than the wall. As a matter of fact they seemed to be useless in this respect. It is possible that in sinking the trenches for these buttresses, the clay settled away slightly from under the wall, and that later water got into the void thus formed, reducing the coefficient of friction there. In any case, as will be seen from Fig. 2, when the earth in front of the buttresses was forced forward by the movement of the wall, the buttresses themselves tilted backward and slid up the slope of rupture. It is evident that this could not have happened if the buttresses had been built in under the wall by underpinning, and probably if this had been done they would have been effective. In fact the author has successfully stopped horizontal movement in a large retaining wall on clay by underpinning it in this fashion.

(4) *Making wall wider.*—A wall design which is suspected of being unstable may often be improved by increasing its width. This affects its stability in two ways. The extra weight of material induces increased friction at the base, and hence increased resistance to horizontal movement. And again the greater width of base gives a more even distribution of load on that base, and consequently reduces the reacting pressure tending to crush the earth under its toe.

It is of interest to notice that the base width of a high quay wall on an earth foundation is invariably much greater in proportion to its height than that of a small wall. Many small walls have been built with a base width equal to one-quarter of the height. But with high walls the widths are much greater. An extreme case is shown in Fig. 6, which represents a very successful and economical quay wall at Montreal.* Its base width is no less than three-quarters of its height. This is necessitated by the lightness of the material of which it consists, viz. rubble stone enclosed in timber boxes or cribs.

It will be realized that in a gravity wall the chief use of the material of which it is composed is to impose weight on the base of the wall, and that the stresses on it are very low.

* See *Minutes of Proceedings Inst. C.E.*, vol. cxcviii.

Consequently the material itself should be of the cheapest kind. This consideration has led to the use of hollow monoliths or caissons of masonry concrete or reinforced concrete, which, when sunk, are filled with sand, rubble stone, or weak concrete. Some excellent walls of this type have been built at Glasgow, Avonmouth, and elsewhere. A cross-section of the Avonmouth wall is shown on Fig. 11.*

(5) *Removing a portion of the backing.*—This is a device which has been frequently used to increase the stability of a wall which has been found to move after construction. An example of it has already been considered (see Fig. 2). Its effect is somewhat difficult to calculate unless the backing

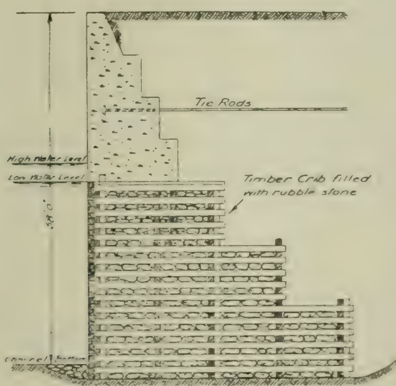


FIG. 6.

is removed right back to the slope of repose passing through the heel of the wall. In the instance cited it did not appear to have much good effect, but there are cases where a wall has been saved from collapse by its adoption, notably at Southampton, where several of the older walls were treated in this way. It has the practical disadvantage that water generally finds its way into the void left by the removed backing, and tends to soften what is left, and thus to increase the pressure on the wall.

(6) *Improving backing.*—A far better device is to substitute a light and clean backing like ashes for the silts and clays which are so often the only materials readily available for

* See *Engineering*, July 3, 1908.

this purpose. Ashes, even when charged with water, weigh only about 100 lbs. per cubic foot, and have an angle of repose of about 35°. Had the Empress Dock Wall been backed with this material, theory would indicate that the horizontal pressure per foot-run would have been 16·0 tons, instead of the 27·8 tons estimated for the sandy clay. One cannot, of course, verify such figures by experiment, but the author knows of a case of a quay wall, about 56 ft. high, and backed with clay, which started to move. The movement

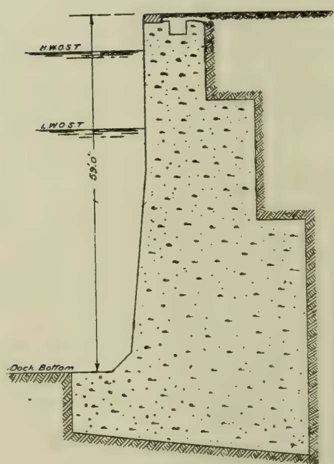


FIG. 7.

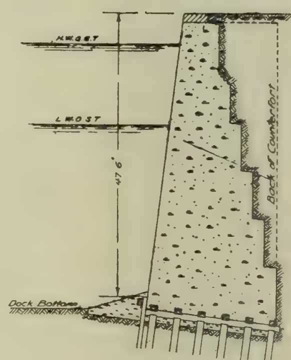


FIG. 8.

was arrested by removing the clay to a depth of about 10 ft. and substituting ashes.

(7) *Sloping base of wall.*—This device is one over which there has been much discussion. Some engineers say that the base of a quay wall should always be sloped from front to back, and others maintain that it is better to carry the foundations down in front to at least the same level as at the back so as to get increased depth of toe. From theoretical considerations we would gather that provided the friction or resistance to horizontal shear were the same at the base of the wall as it is in the earth below, it makes no difference whether the base is sloped or level. In either case if the wall moves it will shear the ground through a horizontal plane passing through the lowest point of the wall.

But as explained above, it seems likely that the friction or shear resistance is greater in the virgin earth than at the plane of junction between the base of the wall and the earth. This being assumed, it is certainly advantageous to slope this plane downwards from front to back, as in that case the wall has either to shear the virgin ground or else move uphill.

Experience of many walls on the sandy clay at Southampton bears out that a sloping base is more stable than a level one. The former style has been adopted for the White Star Dock Wall (shown on Fig. 7), which is the highest quay wall yet built on a clay foundation.

(8) *Driving piles under base.* This is a device which was frequently adopted at one time, though lately it has rather gone out of fashion. Fig. 8 shows a cross-section of a quay wall built at Portsmouth Dockyard * in 1870, and is typical of many walls built during the last century. The wall here shown was quite successful, but there are one or two notable instances of failure with this class of foundation. The fact is, that bearing piles are useful for assisting a weak stratum to bear the weight of a heavy wall, but they are of little help to resist horizontal movement. Probably in most cases, if well driven, the piles take the whole of the weight of the wall. Consequently the friction between the wall and the earth beneath it is nil, and the horizontal pressure from the backing is largely transmitted to the piles, which may fail by ploughing through the earth in which they are driven, and perhaps by breaking under the bending stress thus induced. For this reason and also because they are difficult and expensive to drive inside a wall trench, their use has been largely abandoned. Both theory and experience suggest, that if used they should not be driven plumb, but battered, so that the resultant of the horizontal pressure of the backing and of the vertical weight of the wall should be as nearly as possible parallel to the length of the piles.

Probably if piles are used under a gravity wall, the most effective way of placing them is to drive them as sheeting under the toe of the wall. Fig. 9 is a cross-section of the oldest quay wall at Southampton. As will be seen it is an unusually light one for a quay wall on earth, and its toe is not sunk below dock bottom level. It is not surprising that every length of wall built to this design has moved forward at some time or other. Most of it has been saved from complete failure by removing a portion of the backing,

* See *Minutes of Proceedings Inst. C.E.*, vol. lxiv.

and under these circumstances it has proved to be just about stable and no more. Were it not for the sheeting piles shown on the section, this wall would have undoubtedly collapsed entirely.

Although piles under a high concrete wall are probably of little use, some very excellent and economical designs are to be found in America and on the Continent, formed almost entirely of piles of which Fig. 10 is a typical example. In such a wall the horizontal pressure of the earth backing is transmitted by means of the front sheeting piles, partly to the ground below dock bottom, and partly to the decking which rests on the piles. The decking thus acts as an anchor tie, and is itself prevented from moving forward

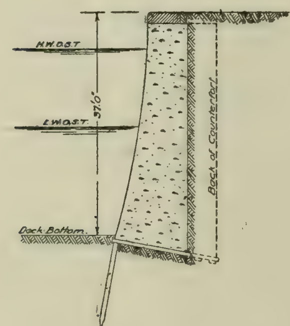


FIG. 9.

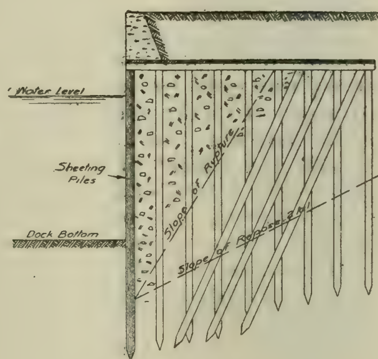


FIG. 10.

by the raking piles under its tail end. The latter piles should of course be driven at such an angle that they are more inclined to a vertical line than the resultant of the horizontal pull on the decking, and of the vertical weight of the earth above it. Besides thus acting as a tie to the sheeting piles, the deck serves to relieve them from the horizontal pressure due to the weight of the earth above it, as the latter is transmitted direct to the piles under the deck. To give this relief effectually, the deck should be wide enough to cover the slope of rupture of the backing, and to nearly reach the slope of repose. The sheeting piles must, of course, be strong enough to withstand the bending moment produced by the horizontal pressure of the backing. To reduce this pressure as much as possible, a specially chosen

backing material, such as rubble stone or clean sand, is usually adopted.

It is probable that the vertical piles behind the sheeting, besides supporting the deck, serve to reduce the horizontal pressure of the earth backing, but to what extent they do so can only be guessed.

The author does not know of any case of failure of this class of wall, so that it is not possible to compare the results of calculation and experience.

(9) *Anchor ties.*—This has been, and is still a favourite aid to the stability of quay walls. In the opinion of many

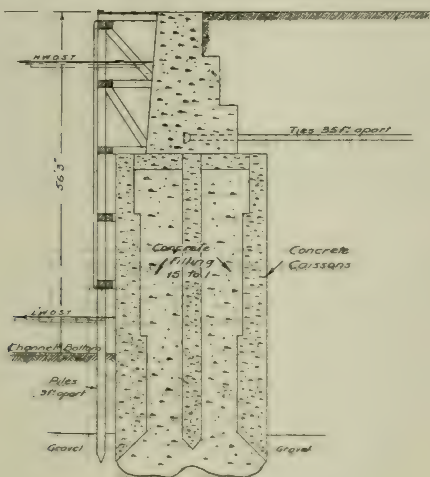
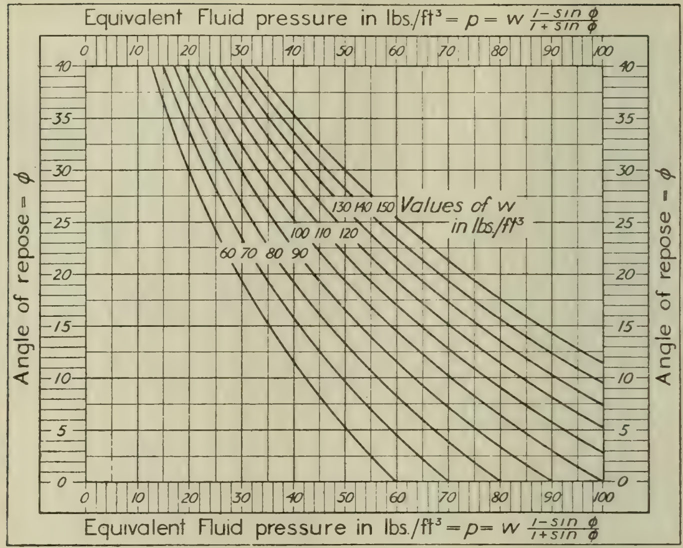


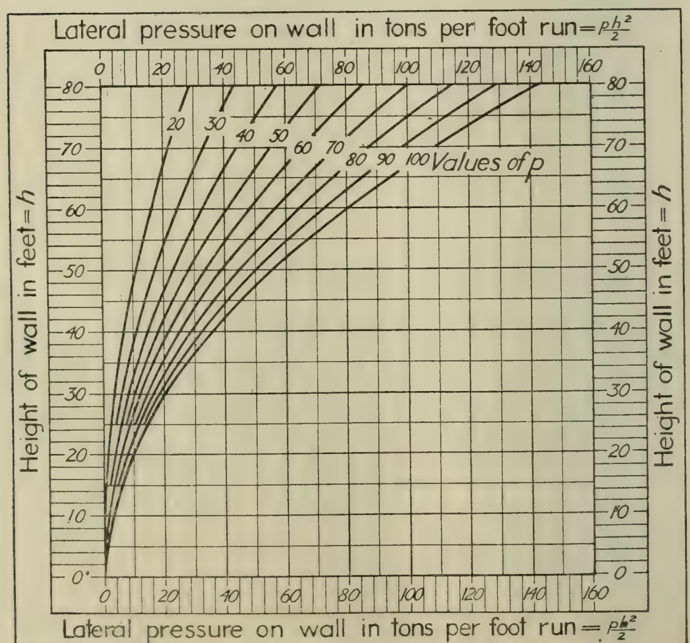
FIG. 11.

engineers, however, they are of little use, partly because they are often placed so far apart that they can only bear a trifling proportion of the horizontal thrust, and partly because the weight of the backing which rests on them, imposes on them a stress which alone is sometimes enough to break them. When the old wall at Southampton shown in Fig. 9 was found to be weak, it was assisted by iron ties $2\frac{1}{2}$ in. diameter placed 30 ft. apart. Even neglecting the stress induced by the backing, this would only act as an inward resistance equivalent to about 1 ton per foot-run of wall. Considering that the outward pressure of the backing must have been about 10 to 15 tons per foot-run, it seems hardly worth while



Equivalent Fluid pressure in lbs./ft³ = $p = w\frac{1-\sin\phi}{1+\sin\phi}$
Angle of repose = ϕ
Values of w in lbs./ft³
60 70 80 90
100 110 120
130 140 150

FIG. 12.



Lateral pressure on wall in tons per foot run = $\frac{ph^2}{2}$
Height of wall in feet = h
Values of p
20 30 40 50 60 70 80 90 100

FIG. 13.

to have gone to the expense of inserting them. As it was, it was found on examination some years after they were put in, that all of them had torn asunder, although the wall was still standing without them! Probably the only case in which they are really effective is that of a jetty with two

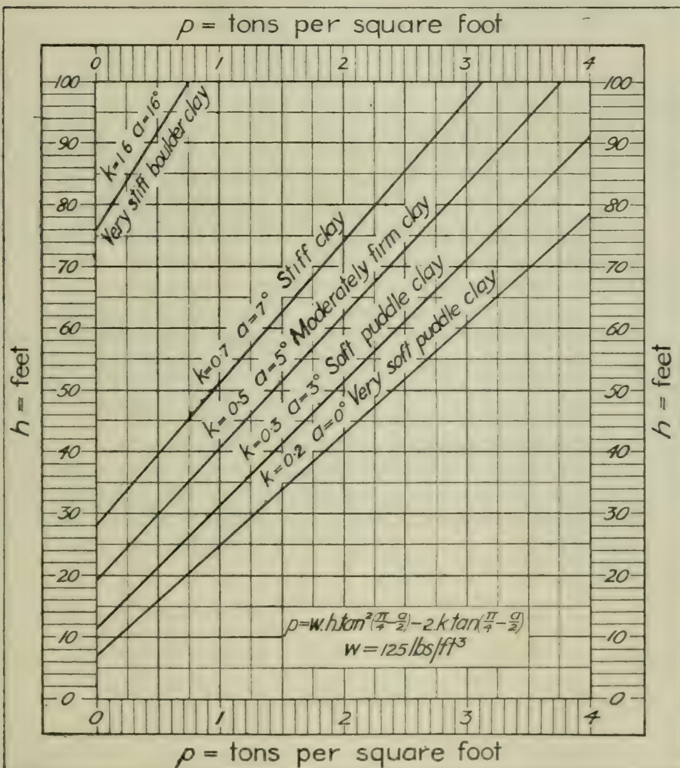


FIG. 14.

parallel walls back to back with earth between them. In such a case the two walls may be economically tied to each other with steel rods.

(10) *Lengthening toe.*—In cases where it is suspected that a wall will crush the material under its toe, the obvious remedy is to lengthen that toe. The White Star Dock Wall shown on Fig. 7 has a toe 9 ft. long which was added in order to

reduce the pressure under this heavy wall to about 4 tons per square foot. In this case the toe was reinforced with old rails in order to prevent its being broken off from the body of the wall.

As previously mentioned it is quite difficult to find instances of quay walls which have failed by overturning, so that the value of this device can only be conjectured.

In conclusion it may be said that owing to the difficulty and expense of making experiments on large walls, it is the more important that careful records should be kept and published of both successes and failures in this class of structure. Such records should include not only dimensions of the wall, and careful notes as to the nature of the materials employed, but also the calculations for stability, with the formulæ used, the assumptions made for the values of constants in those formulæ, and the reasons for such assumptions.

It is of course unlikely that it will ever be possible to design large quay walls purely from rules and without some measure of judgment, but if a body of information, as suggested above, could be collected and analysed, it would help to clear away some of the uncertainties which now beset that interesting though troublesome problem—viz. the most economical design of high quay walls on earth foundations.

The author has to thank the Council of the Institution of Civil Engineers, the editor of *Engineering*, and others who have given permission to reproduce the sections of walls and formulæ given in this paper.

Appendix I.

Empress Dock Wall, Southampton. Calculations for stability. Wall fully backed, and front excavation completed to dock bottom—no tidal water.

To find outward and inward horizontal forces on wall per foot-run.

Pressure of backing from Rankine's formula—

$$P = \frac{w h^2}{2} \cdot \frac{1 - \sin \phi}{1 + \sin \phi}$$

where P = horizontal pressure of backing (tons per foot-run),

w = weight of backing $\left(=\frac{120}{2240}\text{ tons per cubic foot}\right)$,

h = height of wall (= 51·5 ft.),

ϕ = angle of repose (say 26°).

$$P = \frac{\frac{120}{2240} \times 51{\cdot}5^2}{2} \times \frac{0{\cdot}562}{1{\cdot}438}$$

$$= 27{\cdot}8 \text{ tons.}$$

Resistance of earth in front of toe, from Rankine's formula—

$$R = \frac{w\,d^2}{2}\,\frac{1 + \sin\phi}{1 - \sin\phi}$$

where R = resistance of earth (tons per foot-run),

w = weight of earth $\left(=\frac{120}{2240}\text{ tons per cubic foot}\right)$,

d = depth of earth (= 6·5 ft.),

ϕ = angle of repose (= 26°).

$$R = \frac{\frac{120}{2240} \times 6{\cdot}5^2}{2} \times \frac{1{\cdot}438}{0{\cdot}562}$$

$$= 2{\cdot}9 \text{ tons.}$$

Resistance due to friction at base of wall, $R = wf$,

where R = resistance due to friction at base (tons per foot-run),

w = total weight on base of wall (= 62 tons),

f = coefficient of friction between concrete and earth (say 0·3).

$$R = 62{\cdot}0 \times 0{\cdot}3$$

$$= 18{\cdot}6 \text{ tons.}$$

Balance of horizontal forces—

	Outwards. Tons.	Inwards. Tons.
Pressure of backing	27·8	
Resistance of earth at toe		2·9
Resistance due to friction at base ...		18·6
	27·8	21·5

An excess of 6·3 tons per foot-run of wall outwards—unsafe.

Note.—The north wall stood for a short time under these conditions. Probably the friction at the base was at first sufficient, i.e. it amounted to 24·9 tons for a time. Assuming this :—

To find overturning moment, take moments of horizontal forces, say about cope level.

	Clockwise. Foot-tons.	Counter-clockwise. Foot-tons.
Pressure from backing, 27·8 × 34·4	955	
Resistance from toe, 2·9 × 49·4...		143
Friction at base, 24·9 × 51·5 ...		1,280
	955	1,423

Excess counter-clockwise = overturning moment
= 468 foot-tons.

Note.—Moments may, of course, be taken about any other point if preferred—result will be the same.

To find position of centre of vertical reacting pressure from ground under base.

Take moments of vertical forces, say about cope line—

	Clockwise. Foot-tons.	Counter-clockwise Foot-tons.
Weight of wall, 57·5 × 5·1 ...	293	
Weight of backing on wall, 4·5 × 14·0	63	
Friction between backing and back of wall	Neglected	
Reaction under base, 62·0 × x (x = 1·8 ft.)	112	
	468	Nil

Excess clockwise righting moment must = 468 foot-tons.

Therefore centre of reaction is 1·8 ft. from cope = 2·8 ft. from mid-point of base.

To find intensity of vertical reacting presure at toe—

$$r = \frac{W}{B} + \frac{W \times a}{\frac{1}{6} \times l \times B^2}$$

where r = intensity of reacting pressure (tons per square foot),
W = weight on base of wall (= 62·0 tons),
B = width of base (= 30·0 ft.),
a = lever arm, or distance of centre of reaction from mid-point of base (= 2·8 ft.),
l = length of wall considered (= 1 ft.),

$$r = \frac{62 \cdot 0}{30 \cdot 0} + \frac{62 \times 2 \cdot 8}{\frac{1}{6} \times 1 \times 30 \cdot 0^2}$$
$$= 2 \cdot 05 + 1 \cdot 15$$
$$= 3 \cdot 2 \text{ tons per square foot.}$$

To find intensity of vertical reacting pressure at heel—

$$r = \frac{W}{B} - \frac{W \times a}{\frac{1}{6} \times l \times B^2}$$
$$= 2 \cdot 05 - 1 \cdot 15$$
$$= 0 \cdot 9 \text{ ton per square foot.}$$

To find maximum permissible reacting pressure at toe—

$$r = (w\,d + w_{,}\,d_{,}) \left(\frac{1 + \sin \phi}{1 - \sin \phi}\right)^2$$

where r = intensity of permissible pressure (tons per square foot),
w = weight of earth $\left(\frac{120}{2240}\text{ tons per cubic foot}\right)$,
d = depth of earth (6·5 ft.),
$w_{,}$ = weight of water (tons per cubic foot),
$d_{,}$ = depth of water (feet, nil in this case),
ϕ = angle of repose (26°),

$$r = \frac{120}{2240} \times 6 \cdot 5 \times \left(\frac{1 \cdot 438}{0 \cdot 562}\right)^2$$
$$= 2 \cdot 3 \text{ tons per square foot,}$$

as against actual pressure of 3·2 tons per square foot—unsafe.

Note.—An experiment gave the maximum permissible pressure on the surface of this clay as $3\frac{1}{4}$ tons per square foot. The "actual" pressure of 3·2 tons per square foot given above is probably more than really obtains, owing to the friction between backing and wall having been neglected. See Appendix IV.

To find minimum permissible reacting pressure at heel—

$$r = w\,h\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

where r = intensity of permissible pressure (tons per square foot),

w = weight of earth $\left(\frac{120}{2240}\text{ tons per cubic foot}\right)$,

h = height of wall (51·5 ft.).

$$r = \frac{120}{2240} \times 51{\cdot}5 \times \left(\frac{0{\cdot}562}{1{\cdot}438}\right)^2$$

$$= 0{\cdot}4 \text{ ton per square foot,}$$

as against 0·9 tons per square foot (safe).

Appendix II.

West Wall of Empress Dock, with backing removed to $12\frac{1}{2}$ ft. below cope, and with tidal water admitted to Dock—neglecting effect of buttresses in front of wall.

To find outward and inward horizontal forces on wall. Pressure of backing (by Rankine's formula)—

$$P = \frac{w\,h^2}{2}\,\frac{1-\sin\phi}{1+\sin\phi}$$

If the backing were removed down to $12\frac{1}{2}$ ft. below cope as far back as the slope of repose, the pressure would be as follows:—

$$P = \frac{\frac{120}{2240} \times 39{\cdot}0^2}{2} \times \frac{0{\cdot}562}{1{\cdot}438}$$

$$= 15{\cdot}9 \text{ tons.}$$

It was not removed so far back, however, so presumably the value of P lies between 15·9 tons (as above) and 27·8 tons (as obtained in Appendix I). Assume it to have the mean value or—

$$P = 22\cdot0 \text{ tons.}$$

Pressure of tidal water (26 ft. deep at L.W.)—

$$P = \frac{w\,h^2}{2}$$

$$= \frac{\frac{64}{2240} \times 26^2}{2}$$

$$= 9\cdot7 \text{ tons.}$$

Resistance of earth in front of toe (as in Appendix I)—

$$R = 2\cdot9 \text{ tons.}$$

Resistance due to friction at base—

$$R = W \times f$$

$$= 70\cdot2 \times 0\cdot3$$

$$= 21\cdot0 \text{ tons.}$$

Balance of horizontal forces—

	Outward. Tons.	Inward. Tons.
Pressure of backing...	22·0	
Pressure of water		9·7
Resistance of earth at toe		2·9
Resistance due to friction at base...		21·0
	22·0	33·6

An excess of 11·6 tons per foot-run inwards (safe).

Note.—The west wall had a slight tendency to move under these conditions, so that the excess on the safe side of 11·6 tons did not really exist here. Possibly the resistance due to friction at the base became reduced to something like $70{\cdot}2 \times 0{\cdot}15 = 10{\cdot}5$ tons.

Appendix III.

Empress Dock Wall under same conditions as in Appendix I.

To find the horizontal pressure of backing by Bell's formula—assuming a soft puddle clay down to $28\frac{1}{2}$ ft. below cope, and a moderately firm clay below that level.

The general formula for intensity of horizontal pressure at any point behind the wall is—

$$p = w\,h \tan^2\left(\frac{\pi}{4} - \frac{a}{2}\right) - 2\,k \tan\left(\frac{\pi}{4} - \frac{a}{2}\right),$$

where p = intensity of horizontal pressure (tons per square foot),

w = weight of backing $\left(\frac{120}{2240}\text{ tons per cubic foot}\right)$,

h = height from point considered to surface (feet),

a = a constant angle,

k = a constant (tons per square foot),

for soft puddle clay $a = 3^\circ$, $k = 0{\cdot}3$.

The value of p will be nil down to a point h_0 below surface where—

$$h_0 = \frac{2\,k}{w} \cot\left(\frac{\pi}{4} - \frac{a}{2}\right)$$

$$= \frac{2 \times 0{\cdot}3}{\frac{120}{2240}} \times 1{\cdot}05$$

$$= 11{\cdot}75 \text{ ft.}$$

where $h = 28{\cdot}5$ ft.—

$$p = \left(\frac{120}{2240} \times 28{\cdot}5 \times 0{\cdot}95^2\right) - (2 \times 0{\cdot}3 \times 0{\cdot}95)$$

$$= 0{\cdot}81 \text{ ton per square foot.}$$

Just below this point occurs the moderately firm clay where $a = 5°$, $k = 0{\cdot}5$, hence—

$$p = \left(\frac{120}{2240} \times 28{\cdot}5 \times 0{\cdot}916^2\right) - (2 \times 0{\cdot}5 \times 0{\cdot}916)$$

$$= 0{\cdot}36 \text{ ton per square foot.}$$

and where $h = 51{\cdot}5$ ft.—

$$p = \left(\frac{120}{2240} \times 51{\cdot}5 \times 0{\cdot}916^2\right) - (2 \times 0{\cdot}5 \times 0{\cdot}916)$$

$$= 1{\cdot}40 \text{ tons per square foot.}$$

Hence total pressure due to backing—

$$P = \frac{0 + 0{\cdot}81}{2} \times (28{\cdot}5 - 11{\cdot}75) + \frac{0{\cdot}36 + 1{\cdot}40}{2} \times (51{\cdot}5 - 28{\cdot}5)$$

$$= 6{\cdot}8 + 20{\cdot}2$$

$$= 27{\cdot}0 \text{ tons.}$$

To find the resistance of earth in front of toe by Bell's formula, assuming earth to be a moderately firm clay—

$$r = w\,d \tan^2\left(\frac{\pi}{4} + \frac{a}{2}\right) + 2\,k \tan\left(\frac{\pi}{4} + \frac{a}{2}\right)$$

where r = intensity of horizontal resistance (tons per square foot),

w = weight of clay $\left(\frac{120}{2240}\text{ ton per cubic foot}\right)$,

d = depth below surface (feet),

a = a constant angle ($5°$),

k = a constant ($0{\cdot}5$ ton per square foot).

At surface $d=0$—

$$r=2\times 0{\cdot}5\times 1{\cdot}09$$
$$=1{\cdot}09 \text{ tons per square foot.}$$

At base $d=6{\cdot}5$—

$$r=\left(\frac{120}{2240}\times 6{\cdot}5\times 1{\cdot}09^2\right)+(2\times 0{\cdot}5\times 1{\cdot}09)$$
$$=0{\cdot}415+1{\cdot}09$$
$$=1{\cdot}50 \text{ tons per square foot.}$$

Total resistance—

$$R=\frac{1{\cdot}09+1{\cdot}50}{2}\times 6{\cdot}5,$$
$$=8{\cdot}4 \text{ tons.}$$

To find maximum permissible reacting pressure under toe by Bell's formula, assuming earth to be moderately firm clay,

$$r=(w\,d+w_{,}\,d_{,})\tan^4\left(\frac{\pi}{4}+\frac{a}{2}\right)$$
$$+2\,k\tan^3\left(\frac{\pi}{4}+\frac{a}{2}\right)$$
$$+2\,k\tan\left(\frac{\pi}{4}+\frac{a}{2}\right)$$

where r = intensity of permissible pressure (tons per square foot),
w = weight of clay $\left(\frac{120}{2240}\text{ tons per cubic foot}\right)$,
d = depth of clay (6·5 ft.),
$w_{,}$ = weight of water (tons per cubic foot),
$d_{,}$ = depth of water (nil in this case),
k = a constant (0·5 ton per square foot),
a = a constant angle (5°),

$$r = \left(\frac{120}{2240} \times 6{\cdot}5 \times 1{\cdot}42\right) + (2 \times 0{\cdot}5 \times 1{\cdot}30)$$
$$(+ 2 \times 0{\cdot}5 \times 1{\cdot}09)$$
$$= 0{\cdot}50 + 1\ 30 + 1{\cdot}09$$
$$= 2{\cdot}9 \text{ tons per square foot.}$$

Appendix IV.

Empress Dock Wall under same conditions as in Appendix I.

To find effect of considering vertical friction between wall and backing.

Horizontal pressure of backing (see Appendix I)	= 27·8 tons.
Vertical friction due to same, say 27·8 × 0·3	= 8·3 tons.
Overturning moment due to horizontal forces (see Appendix I)	= 468 foot-tons.

To find position of centre of vertical reacting pressure from ground under base.

Take moments of vertical forces, say about cope line—

	Clockwise. Foot-tons.	Counter-clockwise. Foot-tons.
Weight of wall, 57·5 × 5·1 (down)	293	
Weight of backing, 4·5 × 14·0 (down)	63	
Vertical friction at back, 8·3 × 16·0 (down)	133	
Reaction under base, 70·3 × x (up) (x = 0·3 ft.)		21
	489	21

Excess clockwise = righting moment must = 468 foot-tons.

Hence centre of reaction is 0·3 ft. from cope = 0·7 ft. from mid-point of base.

To find intensity of vertical reacting pressure at toe—

$$r = \frac{W}{B} + \frac{W \times a}{\frac{1}{6} \times l \times B^2}$$

$$= \frac{70\cdot3}{30} + \frac{70\cdot3 \times 0\cdot7}{\frac{1}{6} \times 1 \times 30^2}$$

$$= 2\cdot7 \text{ tons per square foot.}$$

To find intensity of vertical reacting pressure at heel—

$$r = \frac{W}{B} - \frac{W \times a}{\frac{1}{6} \times l \times B^2}$$

$$= 2\cdot0 \text{ tons per square foot}$$

DISCUSSION.

THE PRESIDENT (Professor HENRY ADAMS, M.Inst.C.E., etc.) :—The author has set before us very clearly the difficulties, that encompass the engineer, when designing quay walls for a modern deep dock. Many years ago I saw a dock wall being constructed and predicted its failure by overturning, which, however, did not occur until some twenty years later. The original sloping bank behind the wall was simply filled in without being benched, the consequence being, that undue pressure was brought upon the wall. After the failure a Great George Street engineer was called in and a new wall built up of cylindrical casings, sufficient to defy an earthquake and at a proportionate cost. Failure by sliding forward occurred in another case with a properly constructed wall through dredging too near the toe of the wall. The case of the sliding wall of the Empress Dock, described by the author, is particularly interesting, and the only remark I have to make upon it is that, apart from going deeper, I should have made the base slope backwards, so that the wall would have to lift before it could slide forward. With regard to Fig. 2, I do not think, it is quite the right expression to say that "the wall

overturned backward about its heel." It seems to me that, in sliding forward, the weight of the wall practically rested upon the heel, which therefore dug into the foundation. The thrust being greatest towards the bottom, and the line through the centre of gravity of the wall being behind the centre of its base, the natural effect was to push it forward more at the bottom than at the top. Another explanation of the failure in the cases of both Fig. 1 and Fig. 3 occurs to me, and that is, that the pressure of 3·2 tons per square foot at the toe upon the moist earth at the bottom of the dock caused the material to spew up in front and so reduce the resistance to sliding. I may say that I always use Rankine's formula for retaining walls, even for clay; but the main question is, "What is the natural slope for clay?" In my experience, when clay gets moist it acts more like a viscid fluid, and it is hardly possible to say, that it has any real natural slope. You will have seen by the reference to the friction on clay that, when it is in a dry state, it is virtually on a different material altogether. The reference to Mr. Bell's paper reminds me, that I have always considered the line of rupture as more like a semi-parabola with the vertex at the surface, as will be seen in any section immediately after a slip has occurred. In the case of water getting behind a wall, Sir Benjamin Baker shewed, that the pressure was greater than if the backing were water only; I think the figures, he gave were one and a half times that of the dry material. I certainly prefer Rankine's formula for the pressure upon a foundation to that of Bell, quoted by the author on page 183. Another point generally overlooked is, that if the water penetrates under the wall, the wall tends to float in proportion to the specific gravity of the water compared with that of the wall. Anchor ties were used in between a pair of quay walls at the Victoria Docks, London, and movement was found to be in progress from the ties rusting through. That was discovered just in time to prevent a disastrous failure. In conclusion, I should like to express my opinion that this is the most important engineering paper, that has been read before the Concrete Institute, and I congratulate the author upon

having given us such a complete résumé of the subject.

MR. CHARLES F. MARSH, M.Inst.C.E., M.Am.Soc.E. (Vice-President C.I.) :—I did not think, there was really much good in sloping back the foundation of a wall for the reason which Mr. Sheilds gave, that the clay itself would fail in shear owing to the horizontal pressure behind the wall, and the sloping back really did not, in my opinion, help very much. In his personal experience he knew two very well-defined cases of failures of walls by sliding forward. They were reservoir walls in London clay, and the failures, curiously enough, occurred within four days of each other. There was no sign of any overturning, and not only the walls slid, but, in one case, an entire bay, and in the other case the cross walls and footings for the length of one bay, these portions of the work having been built in front of the wall, as is the ordinary practice, to hold the walls steady, till the floor was completed. In both cases the whole constructed portion moved forward bodily, and the cracks through the walls did not follow the joints, but cut through the bricks, both stretchers and headers. The bricks in one case were hard-burnt reds, and in the other Staffordshire brindles. The photographs show the nature of the failures. The sliding forward of the walls in both cases appeared to be due to the backing behind the wall being brought up to its full height, before the wall had received the support of the floor, and also to the fact, that a crane was working on a road immediately behind the wall. In both cases the walls were reconstructed without any alteration being made to the designs, and no further trouble was experienced. The rainfall just prior to the dates of failure was not in any way abnormal. I think, there is no doubt, that when a big wall was designed with a clay backing and on a clay foundation the main thing to look after was the sliding forward at the base. If any considerable wet weather occurred during the construction of the wall, there was no doubt, that the water got down behind or round the wall and soaked in underneath, it being frequently found that there was practically no co-

efficient of friction at all between the wall and the foundation. Recently a reservoir had to be designed in the London clay, and it was decided, that instead of specifying, that a bay should be built with the wall, so as to prevent failure before the floor and roof were constructed, that a tongue should be taken down underneath the wall. As a matter of fact, the wall was about 17 ft. high, and the tongue was taken down 10 ft. and was made 3 ft. wide. It was reinforced with rolled joists vertically and with bars horizontally. The reservoir has not yet been completed, but he believed that the wall when constructed would be perfectly safe, as the tongue would hold the wall from silding forward. Mr. Sheilds remarked, that with regard to the design of dock walls he would have thought, that they should have been designed without taking any account of the water in front, because it was obvious, that the wall must be constructed before the water was let in, and, as in the case of the Empress Dock, the wall, if it was going to fail, would do so before the water was let in. With regard to the angle of repose of clay, I think that 26 degrees is rather a steep angle, and prefer about 15 degrees, because of the treacherous nature of clays.

Mr. M. E. Yeatman remarked on the uncertainty of the amount of pressure on the back of a wall, where water had access to the filling. He considered, that to take the full pressure due to the head of water, and add that, due to the filling taken at its weight in water multiplied by the coefficient due to its angle of repose, was correct for an open, pervious filling like rock rubble, but would probably give an error on the safe side in the case of materials like sand, where the water being held up by capillarity would not exert its full pressure.

He advocated the use of rough-bottomed walls, where they had to be built on a clay foundation, and the danger of sliding forward was greater than that of overturning. This rough bottom could be provided by spreading a layer of coarse rock rubble over the levelled foundation and driving the stones down about half their depth into the clay; then

concrete can be laid around and over these stones, which will form a bond between the concrete and the clay and prevent sliding.

DR. J. S. OWENS, B.A., M.D., Assoc.M.Inst.C.E., M.R.San.I., F.R.G.S., F.G.S. (Member of Council C.I.) :—Referring to page 186, in the first paragraph, Mr. Sheilds refers to the vertical friction of the backing, tending to prevent the overturning of the wall in Fig. 2. Looking at the section of the wall in Fig. 2 on page 177, it seems to me, that the vertical friction is operating the other way—that is, to oppose the movement. Apparently the movement of the wall was downwards relative to the backing, so that the friction would be acting upwards, tending to lift the back of the wall instead of pressing it down, which I think is assumed in that paragraph.

It seems to me, that the whole crux of the problem is the determination of the constants, such as the angle of repose of materials, and it is quite impossible to assign, with accuracy, angles of repose to certain materials. There are only some, such as shingle, which do not alter their angles of repose with their moisture content, and which you can really make any sound calculations about, because if you take a material like clay, the angle of repose varies with the moisture present. Another thing, not referred to in the paper, is the effect of vibration, and I mention it for the sake of completeness. The angles of repose of materials appear to alter, according to whether they are subject to vibration or not, and that perhaps is connected with the difference between adhesion and moving friction. I noticed with interest Mr. Marsh's description of the two walls, which failed almost simultaneously. I do not know, but suggest that it was also simultaneous with very heavy rain or vibration or something altering the internal friction of the backing. With regard to the slope at the base of the wall, I think Mr. Sheilds has made that very clear. If the coefficient of friction between the base of the wall and the clay is greater than that between clay and clay, then there is no object in sloping the base of the wall. On the other hand, if it is less, slope the base. One other little point occurs to me.

Looking at Fig. 2, I was reminded of a paper read by Dr. Vaughan Cornish on the movements of clay in the Panama Canal excavations. One of the curious things, he described, was a sort of wave movement, in which a settlement took place at one point, then there was a considerable interval where the surface was quite unaffected, and beyond this there was an uplift. These uplifts kept occurring in the cuttings of the Canal and caused a lot of trouble. It suggests to me, that some of the failures of retaining walls might be elucidated if a wider view of the thing were taken. The clay for some distance around may be undergoing some curious movements, because when you take out a big excavation you relieve the pressure, and you may get all sorts of queer movements taking place due to Nature's efforts to establish equilibrium. It is conceivable, for instance, that the site of the wall might behave as a node, and settlement take place at the back with an uplift in front of the toe. I do not think, that is often the case, but it is worth keeping in mind. I would like to join with the other members in thanking Mr. Sheilds for his very valuable paper. It is a most useful type of paper to bring before the Institute.

Mr. S. Bylander (Past Chairman J.I.E.), M.C.I., referred to a wall for which he had been responsible, 60 ft. high, something similar to what the author had placed before them, but an ordinary retaining wall for a building, and not a dock wall. The author, he said, has mentioned, that the horizontal pressure was taken up by the concrete basement floor across to the other side, a similar wall. Continuous reinforcement was placed in the outside portion of the wall. The greatest difficulty was during the construction of the wall, and not afterwards. During the excavation and strutting a slight movement of earth took place, and I found that the clay slid inwards. I observed a little crack about ¼ in. wide in the paving of the area to the house opposite, before the concreting had progressed very far. On the right was a building at least four stories high. The wall was completed, and after the concrete had set sufficiently, the strutting was removed, and after that date no further move-

ment took place, showing that the wall actually resisted the earth pressure. The load of the external walls was placed on the base of the retaining wall, which consequently also added to the stability. Reinforcement was placed in both directions, so that the bottom part of the wall formed a raft. In my experience of designing a retaining wall I have come to the conclusion, that for relatively very low walls you must add surface load, because several failures have occurred with very low walls by adopting an ordinary formula, so I propose, in view of what several speakers have said to-night, and the uncertainty with which we are faced in considering a theoretical pressure on a wall, that a much more simple formula be adopted. When we have to determine the constant applicable to each particular case in each material, I think the formula should take this form :—

$$H = k_1 h + k_2 h^2$$

where H is the total horizontal pressure in pounds per foot run of wall and h is the height of wall in feet.

MR. PERKINS :—Does that take into account the weight of the earth itself, whether it is 90 lbs. or 100 lbs. per cubic foot?

MR. BYLANDER :—The constant varies with the weight of the earth and the angle of repose. That is a practice which need not be varied very much for ordinary conditions.

MR. MARSH :—What relation has k_1 to k_2?

MR. BYLANDER :—I have not got my notes with me, but I think the values are something like this :—

$$100 h + 15 h^2 = H$$

That will give a very reasonable result for ordinary walls up to, say, 25 ft. high and next to main streets with ordinary traffic.

MR. MARSH :—That is in pounds, is it?

MR. BYLANDER :—Yes. I suggest this as average values for London ; it is advisable to agree on some form like this.

MR. MARSH :—Is that an experiment?

MR. BYLANDER :—No, but arrived at, after comparing work executed and found satisfactory.

MR. PERKINS :—Does it take into account a water main bursting in the street and creating a hydraulic pressure? That is not a joke ; it has happened twice to me.

MR. BYLANDER :—No. I also can relate some failures from that reason, and not that reason alone ; but I may say, that in this case a water main burst and no sign of failure has occurred.

MR. MARSH :—100 *h* is a tremendous addition to make to Rankine's formula.

MR. BYLANDER :—Well, try it for low walls ; it works out reasonably. You must allow for street traffic.

MR. MARSH :—Did you get the weight of the piers on to the toe, before you took the timbering of the trenches away?

MR. BYLANDER :—No, the building itself was not erected, until the wall was completed and the strutting removed.

MR. MARSH :—That was no good, because the wall would fail.

MR. BYLANDER :—The wall was strong enough to withstand the pressure without the additional load. The toe of the wall served two purposes : firstly, to give stability to the wall, and, secondly, to form the foundation for the building. I will now show you a case of a dam, 15 ft. high, something similar to what Mr. Marsh mentioned. The question in my mind is, to what extent the water will travel underneath the wall. I think it requires very serious consideration. The proper thing to do is to form a heel or tongue, so as to seal up the water as far as is possible.

MR. MARSH :—Would it not be better to take a tongue down?

MR. BYLANDER :—I don't think so.

MR. W. G. PERKINS (District Surveyor for Holborn, Member of Council C.I.) :—I was, when Mr. Bylander spoke, going to suggest, that a heel should be put at the back of the wall. That seemed to me a good thing for two reasons. The earth under the water-line in front of the wall gets soaked and becomes in the condition of mud. Therefore, for a certain depth down, according to how far the water penetrates, the toe ceases to be of any use, to prevent the wall sliding forward. If you put a tongue or heel at the back, where perhaps the water may not penetrate, the earth would be able to offer a greater resistance to the wall moving forward. You have also the weight of the wall acting down on the earth and compressing it, which would help the earth to resist the tendency of the wall to move forward, and the horizontal component of the pressure would become distributed as it travelled forward. I wanted to ask Mr. Sheilds whether, after he had made his excavation and found his foundation level, he found any springs of water in the earth, because if so you would at once have the water tending to lift the wall in the way suggested by the President; it would also make the clay slippery. Can Mr. Sheilds also tell us, whether there were any contractors' lines of railway upon these quay walls, and if so did he take into account the effect of vibration from the trains?

MR. W. A. GREEN, M.A., B.Sc.Eng., Assoc.M.Inst.C.E., also spoke.

MR. WENTWORTH-SHEILDS, replying to the discussion, said :—The discussion has clearly brought out two facts: (1) that retaining walls generally fail by sliding forward; (2) that no method of calculation of the stability of retaining walls has yet been agreed upon by engineers, chiefly owing to the difficulty of ascertaining the actual lateral pressure of earth backing and the actual friction between the

wall and the earth beneath it. Some speakers like Dr. Cunningham seem inclined to regard these as unsolvable problems ; I think, however, that this is too pessimistic, and that the values of these factors can even now be ascertained within limits, and that further investigations will reduce these limits.

The actual lateral pressure of clay is difficult to obtain from Rankine's formula, owing to the uncertainty of the value of φ. This angle could not be obtained merely from observing the natural slope of a given clay, as even a weak clay will stand upright for a small depth, and when it slips, the surface of cleavage is not a plain one. In this respect Mr. Bell's formula is far more satisfactory.

One or two members have referred to the methods of calculations proposed by Mr. Meem and Mr. Hawes in the *Proceedings of the American Society of Civil Engineers* (vol. xxxiii, 1907). These are certainly most interesting, but I do not think they promise such a satisfactory solution of the difficulty as Mr. Bell's formulæ. Both the American engineers believe, that the centre of lateral pressure of earth is nearer the surface than Rankine's formulæ suggest. I do not think, that facts bear this out. Were it so, one would expect walls to overturn much more frequently than they do. The theory appears to be supported by the fact, that the timbers at the top of a trench are often more stressed than those below. But it must be remembered, firstly, that in most trenches the lower material (especially if clay) is often more cohesive than the upper, and, secondly, that the upper material has always been exposed for a longer period than the lower, and is therefore weaker. In the long run, I think that Rankine's conclusion, that the pressure increases as the depth, is not far wrong.

Some interesting remarks were made about the factor of safety. Mr. Marsh considered that 1·1 would be sufficient ; Mr. Waldram suggested 1·2. It seemed to be agreed that 3·0 was Utopian. Mr. Marsh said that the pressure of water in front of a wall should be ignored, as during construction there is no water in front. This is not always the case, however. Some quay walls are built in water and some in deep trenches, the earth in front of them

being allowed to remain until water is admitted to stabilize them.

The discussion has brought out a consensus of opinion in favour of sloping the base of a retaining wall. Mr. Marsh opposes this device, apparently on the ground that the coefficient of friction between the earth and the wall is the same as that between earth and earth, and that the wall will therefore move on a horizontal plane through its lowest point. As mentioned in the paper, however, this assumption is unlikely to be correct in most cases.

Mr. Yeatman and Dr. Owens referred to the curious clockwise overturning movement of the Empress Dock wall, shown in Fig. 2, page 177. Dr. Owens suggested, that the movement of the wall was downwards relative to the backing. I am inclined to doubt this, and would rather say, that the earth as it fell dragged the back of the wall downwards, and that this partly caused the clockwise movement. Another cause for this movement is, I think, the fact that the toe of the wall as it moved forward travelled up the slope of the rupture, formed by the moving earth in front of it. The tendency of the earth to drag down the back of the wall in this instance justifies the contention, that the friction between the earth backing and the wall may be regarded as helping to prevent the wall overturning in the ordinary sense—that is, forwards about its toe. The President thought, the failure of this wall might be due to the material crushing under the toe; but if that were so, one would have expected a counter-clockwise movement instead of what took place.

Mr. Perkins and others suggested, that a downward projection at the heel of a wall is better than at the toe. With this I quite agree. Mr. Perkins asked whether springs occurred in the foundations at Southampton. The answer is yes; in the sand, but not in the clay. He further asked if the effect of vibrations from railway lines had been considered. Vibration has not been taken into consideration in the calculations, although it is quite likely that it has some effect upon the stability of the wall.

Mr. Waldram asked whether the anchor ties in the old dock wall were rusted. The answer is yes, to

a slight extent, but not sufficiently to account for their failure. He also raised the interesting point as to the effect of the weight of a building sufficiently close to a retaining wall to come either within the plane of repose or within the plane of rupture. Presumably a building which lay between the plane of rupture and the wall would exercise its full effect as a surcharge on the back of a wall. If behind the plane of repose, its effect on the wall would be nil, if between these two planes its effect can only be surmised. I was very glad, that Dr. Owens emphasized the importance of making further experiments.

Finally, I should like to express the thanks of the Institute to Mr. Marsh and to Mr. Grant, who have given us details of some walls that have failed, and I would again appeal for descriptions of other failures. Such descriptions need only be brief ; but besides a dimensioned cross-section of the wall the record should include careful notes as to the nature of the earth backing and of the foundation on which the wall rests. Also if possible a sketch showing the position, which the wall took up after failure. I should like to see a committee of the Institute appointed that would lay down rules for the calculation of the stability of retaining walls. Such rules would no doubt need revision from time to time, but their existence would help to clear away some of the uncertainties, which beset the subject at present.

CORRESPONDENCE.

Contributed by MR. BOYSSON CUNNINGHAM, M.Inst.C.E.

The subject so admirably treated in this paper is one of the first importance to all structural engineers, and particularly to those engaged in dock work. No one who is interested in retaining walls in any shape or form can read the paper without a sense of obligation to Mr. Wentworth-Sheilds for so clear and explicit a statement of the problem and its present unsatisfactory position. It cannot fail to be recognized that, with so many indeterminate factors, it is not possible

to arrive at any exact, or reliably precise, solution. At the very outset, we are called upon to accept the principle of an angle of repose and an angle of rupture, each of which consists of a straight line inclined to the horizon; but what practical man would agree, that such lines represent the facts of the case? Is it not common knowledge, that the line of unsupported cleavage more often assumes the form of a parabola, and that it will vary with the nature of the strata through which it passes?

Even if we accept the convention, who can determine the value of these elements with the least degree of confidence? The experimentalist says that the angle of repose for "well-drained clay" is 45 degrees; but what percentage of dryness or moisture constitutes "well-drained clay"? and who will venture to define even so well known a substance as "clay" in terms which will admit of no erroneous identification between the limits of "loam" and "marl"? "Dry sand" is a term which seems explicit enough; but we have one authority giving the angle of repose as 22 degrees and another authority giving it as between 31 degrees and 37 degrees. No doubt each is correct in regard to the material he experimented with, but the use of a single term to cover substances so widely divergent in their properties cannot fail to confuse and bewilder those who seek to make use of such data.

These discrepancies and uncertainties are well known, and there is no occasion to enlarge upon them here. They are simply recalled to indicate the tremendous and wellnigh insuperable obstacles in the way of any satisfactory solution of the problem. And yet the attempt has to be made somehow. However imperfect, some system of reasoning is imperative: otherwise the practice of engineering would rest upon a foundation of the flimsiest character. And so it is that engineers feel their way as best they can, moving cautiously and leaving a wide margin to cover many and great possibilities of error, in the hope that the ultimate result will, at any rate, be "on the safe side."

With the theoretical aspect of the question in so unsatisfactory a state, it is perhaps pardonable to con-

tent oneself with some reference to the practical side of quay wall construction.

One of the most important practical points to be borne in mind in connection with retaining walls is that of efficient drainage at the back. It is often assumed that in the case of quay walls the water may pass beneath the wall and rise behind to the same height as in front. This is not the worst aspect of the matter. Owing to a falling tide, or from other causes, the water-level may be considerably higher at the back of the wall than in front, with the result that the pressure of the earth backing becomes transformed into hydrostatic pressure of increased density, due to the inherent weight of the material. Moreover, by reason of percolation from the quay surface under continuous or prolonged rainfall, it is quite possible for the earth backing to become saturated right up to coping level. Several cases have come to the writer's knowledge of walls, in other respects sound, which have failed under this adventitious fluid pressure. One was the case of a lock wall, $32\frac{1}{2}$ ft. high above the invert, which leaned outward at the top, quite suddenly, many years after completion. The amount of overhang was 8 in. in the centre of a length of 356 ft., and a week or two later the coping went forward another 3 in. Another was a quay wall, 36 ft. from coping to dock bottom, forming one side of a wet dock, which was in course of being emptied for the purpose of underpinning operations, until the wall showed symptoms of overturning about its toe. The movement in both cases was found to be entirely due to the accumulation of moisture in the material at the back of the wall. The remedy consisted in drilling a series of drains, or weep-holes, through the walls from back to front so that the water could escape. At the same time, the sodden material behind the wall, which was little better than "slurry," was removed by excavation and replaced by selected rock rubbish and good dry filling, with rubble and stone round the apertures of the weep-holes.

It is the writer's conviction that a sloping base, with the heel of the wall deeper than the underside of the toe, is generally a safer and more satisfactory

arrangement than a level base. Whatever the chances of a horizontal shear through the foundation stratum, the balance of advantages undoubtedly lies with the sloping base, except in such cases as walls founded on rock and gravel, where the benefits are less marked. It is a sound principle, which makes the surface of the foundation perpendicular to the line of the resultant pressure upon it, and, however faultily it may be determined, the resultant in the case of an ordinary retaining wall is always inclined in some measure to the vertical.

SIXTY-THIRD ORDINARY GENERAL MEETING

THURSDAY, MAY 20, 1915

THE SIXTY-THIRD ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held in the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, London, S.W., on Thursday, May 20, 1915, at 7.30 p.m.,

MR. CHARLES F. MARSH, M.Inst.C.E., M.I.Mech.E., M.Am.Soc.C.E. (Vice-President C.I.), in the Chair.

The following was elected :—

MEMBER.

MR. WILLIAM FENTON ANDERSON, A.R.San.I., Lecturer in Building Construction, Quantities, Structural Engineering, etc., Head of Building Trades Dept., Bolton Technical School.

MR. E. A. W. PHILLIPS, M.Inst.C.E., M.C.I., Retired Superintendent Engineer, Burma Public Works Department, then read extracts from his paper, entitled "Lime Concrete in the East," the full text of which is appended.

LIME CONCRETE IN THE EAST.

PART I. ORIGINAL PAPER.

STONE lime of great purity, and consequently non-hydraulic, is used largely in India and Burma, and engineers have learnt to place considerable confidence in the material. To enable it to set under water, it is mixed with "soorkhee," the homra of Egypt.* To the present day engineers in India do not know exactly how much soorkhee is required by each

* "Soorkhee" in India is finely powdered red brick.

kind of lime, and this ignorance is due to the want of scientific laboratory tests, of the kind so frequently made in Europe. It seems no advantage to send lime and soorkhee to England to be tested, since the difference in climate, the sea voyage, and the lapse of time in transit might vitiate the results.* Conservative Indian opinion, based on long experience, approves of a mixture of a half-part of under-burnt with a half-part of well-burnt soorkhee to one part of slaked lime and one part of sharp, clean sand, all measured in bulk, dry. The materials are thoroughly incorporated and ground in a mortar-mill, either under one wheel pulled round a circular trough by a bullock, or in a pan-machine under a pair of wheels. The mortar should be a thick reddish paste, in which the particles of lime cannot be distinguished by the naked eye. A mortar made in this way sets very well indeed in still water, but it sets comparatively very slowly, and some engineers (the author included) add, when necessary, a proportion of Portland cement to the mixture. The introduction markedly hastens the setting to an extent depending on the proportion of cement to lime. One part cement to one part lime by volume sets apparently as quickly as cement mortar. In the early stages of setting the strength of the concrete is much increased, admitting of early handling and removal of moulding boards. The addition of cement preserves soorkhee mortar in wet foundations from the evils of percolation, and the cement, besides, seems to have a chemical (?) effect on the lime, fixing the particles and aiding in a more solid set.

First-class soorkhee mortar, several centuries old, it has been asserted, exceeds Portland cement mortar, 1 to 3, in strength and impermeability, and is said to be often equal to 1 to $2\frac{1}{2}$. In regard to economy there is no comparison in Rangoon, cement costing (1909) about two rupees per cubic foot, and lime only three-tenths of a rupee per cubic foot. Further, in comparing the two classes, it has to be remembered that while native Indian masons are exceedingly clever in dealing with lime, they often make bad mistakes in dealing with Portland cement, and have to be

* Bricks and lime-stone might, however, be sent, if carefully packed in air- and water-tight cases.

specially taught its use, and carefully watched besides, as they are stupidly stubborn.

Where a mortar mill cannot be obtained, or when the work is too small to justify the expense, stone lime is thoroughly slaked by being steeped for from twenty-four hours to a week or more in fresh water, being kept constantly covered and stirred up at intervals with a rod. The result is a perfectly slaked lime, and a breaking up of all but the very hard particles. The wet lime is first mixed thoroughly with the sand, the particles of the sand tending to still further break up and subdivide the lime. Soorkhee, very finely hand-pounded and screened, is then added in small quantities, and if the operation is properly done, it should be a little difficult to detect that the mortar has not been prepared in a mill. This is the best native Indian practice when a mill cannot be got. Tradition asserts that the stone lime in the Táj Mahál of Agra was steeped in water nearly a year before use.

A suggestion has been made in *Indian Engineering* to apply the term "béton" to mixtures with both cement and lime, reserving the term "concrete" for the usual mixtures. The usual classification for cements and limes is :—

1. Portland cement.
2. Natural cement.
3. Puzzolan cement.
4. Hydraulic lime.
5. Common lime.

With all the above, concretes can be formed, either hydraulic or non-hydraulic, and it is proposed that the term "béton" be reserved for all mixtures of 1, 2, or 3, with either 4 or 5, in forming mortar.

The author has employed the following mixtures in practice, obtaining exceedingly good, reliable, and economical results in all sorts of places, and especially in bad foundations :—

Béton, 1 : 1 : 3 (Equivalent).

1 part cement
1 part lime
1 part soorkhee } all measured dry.
1 part sand
6 parts aggregate

Béton, 1 : 2 : 4 (Equivalent).

1 part cement 1 part lime 1 part soorkhee 3 parts sand 8 parts aggregate	all measured dry.

Béton, in damp foundations (1 : 3 : 5).

1 part cement 3 parts lime 3 parts soorkhee 5 parts sand 20 parts aggregate	all measured dry.

In all the above the author employed milled mortar if possible, sifting the cement in last of all, and thoroughly incorporating it with the mortar, as shown by uniformity of change in colour.

In above-ground work, where the free lime will be air-set, the soorkhee might be omitted, in which case two mortars might be made, one lime and sand, and the other cement and sand, in any required proportion, and mixed together.

It is not quite certain, however, that such mixtures will not be set hard even under water. The author has made no experiments to prove this.

In dry climates it is essential to keep hydraulic concrete drenched with water, and covered from the direct rays of the sun. The loss of strength in hastily dried concrete or béton is *enormous* ; and nothing is so easy, in the East, as to destroy good material in this way. In the dry season, in the tropics, it is as necessary to take as much care of concrete and to protect it as carefully as a young lady should care for and protect her complexion. The author speaks feelingly from experience, as he has seen so much work, executed with first-class material, in a thoroughly workmanlike manner, ruined because the contractor grudged a trifling expenditure in keeping it wet. Béton especially needs slow and long seasoning, and six months is not too long a period in making blocks of the 1 : 3 : 5 mixture given above. The blocks should be removed from the moulds as soon as they can be

handled (twenty-four hours usually), and placed at once under wet earth or sand, kept at all times wet for six months if possible, and three months for certain.

The worst form of unskilful handling met with in India is over-ramming. The author has dug up brick concrete literally rammed to sand and soorkhee, the mixture having lost all cohesion. In other places, he has found cement concrete fissured and useless, due to ramming prolonged beyond the time of setting. The idea seems to be, in such instances, that concrete should be laid in 6-in. layers, *rammed down to* $4\frac{1}{2}$ in. *or* 5 in.

Another error is the laying of concrete too dry. With all deference to received opinion, the author does not believe in the practical value, in India and Burma, of concrete laid nearly dry and thoroughly rammed. The practice may give excellent results in a laboratory, but on work it is bound to fail. The best mixture is a jelly-like mixture, which can be freely shaken by light ramming, all air displaced, and the material readily and well settled. Heavy ramming is an absolute mistake. The concrete or béton should be laid in small quantities and punned into place, *not* rammed. If too wet, the cement and lime are liable to displacement ; if too dry, the ramming must be unduly prolonged, while it is more than probable, in a dry climate, that there will be insufficient water in the body of the concrete or béton for proper hydration. External watering should be only necessary to prevent evaporation. If the water has to soak in, it will do more harm than good, at least in the early stages of the setting. The risks are by no means imaginary, as the author has found to his cost. It is the easiest thing in the world to turn out bad concrete with the best material and with extreme carefulness. Zeal and watchfulness on wrong lines do far more harm than good. The matter has been fully discussed in *Indian Engineering*, and there is no more to be said about it from the author's point of view.

As already pointed out in *Indian Engineering* and elsewhere, brickwork is worthy of a concrete man's attention, since it is really a hand-packed concrete of a superior quality, the pieces of aggregate, all of equal size, being laid in good bond to secure maximum

strengths of both bricks and mortar. Brickwork should be as superior to brick concrete as a skilled bricklayer is superior to a common labourer. Until this view is accepted and acted upon, brickwork must always be regarded with suspicion. It is obvious that bricks should be made capable of very firm attachment to the mortar employed, and this is an important matter generally lost sight of. Glazed bricks, from a concrete point of view, are about as useful as glazed marbles for aggregate, whatever their value in sanitation may be.

Native Indian bricklayers, left to themselves, employ a first-class mortar, with small, well-burnt porous red bricks, 7 in. or 8 in. long, and from $1\frac{1}{4}$ in. to $1\frac{1}{2}$ in. maximum thickness. Walls are built by running brick ledges along the faces, and allowing them to harden. Mortar is thrown into the trough between, spread, and the interior bricks pressed down till all the vertical joints are filled from below. Bond is very little regarded, but most of the bricks are laid as stretchers. The walls are finished off by a special plaster, sometimes $1\frac{1}{2}$ in. thick or more, made of mortar mixed with grit or hard gravel (budg-ree), and practically a fine concrete. Even here the common fault of insufficient watering often shows itself, and in many cases the plaster fails to attach itself firmly to the walls, the latter having been allowed to dry thoroughly, and being insufficiently wetted afterwards, sucking in moisture and preventing adhesion.

A European adaptation would be to form the walls, much as above, between moulding boards, the plastering and brickwork being cast in one piece. With a half-lap (half the length of a brick) bond, laid roughly to vertical lines marked on the insides of the boards, and with good mortar and good red clay bricks, very good and economical work might be got into retaining walls and dams of moderate height. Since the bricks would not be seen on faces exact bonding would be unnecessary. So long as the bricks overlapped by at least one-third their length, sufficient strength would be secured. [Well-bonded brickwork in which the bricks are tough, and attach themselves firmly to the mortar, is really a form of reinforced concrete.]

Dated: RANGOON, BURMA, *April* 24, 1909.

PART II. EXPERIMENTS.

(Made by Mr. H. Kempton Dyson and the Author at the London County Council School of Building at Brixton.)

INTRODUCTION.

Mr. Dyson is responsible for the appearance of this paper, as he dug Part I out of the Institute records, and insisted on the author proving his assertions. The Associated Portland Cement Manufacturers (1900) very kindly interested themselves in the matter and gave material assistance, without which these experiments could not have been undertaken. The officials of the London County Council School of Building, through Mr. Dyson, were most courteous, and placed their instruments, etc., at the author's disposal. Everybody has helped, and the author can only express his deep obligation to Messrs. Shillito, Cox, Richards, Sage, Dyson, and others.

It seems the Associated Portland Cement Manufacturers have already experimented with mixtures of lime and cement, and of lime, cement, and sand. The results are most interesting, and were communicated to the author in the following letter :—

Extract of Letter dated July 28, 1914.

As promised on the telephone, I have been looking up our records in regard to the use of hydrated lime in Portland cement concrete, from which I have been able to summarize the following information :—

Effect upon Setting Time.

	Initial.		Final.	
	Hrs.	Min.	Hrs.	Min.
Neat Cement	1	20	5	30
99·5 parts cement, 0·5 slaked lime	1	15	4	55
99·0 parts cement, 1·0 slaked lime	1	12	4	52
98·0 parts cement, 2·0 slaked lime	1	5	4	50
95·0 parts cement, 5·0 slaked lime	1	2	4	47

From these figures you will see that the addition of hydrated lime appears to have little effect on the setting time of cement (only slightly quickening it), and these results are confirmed by others which I need not repeat here as the above are fairly representative.

Tensile Strength of Cement Mortar with Various Proportions of Lime added.

3 parts sand to :—	Lbs. per square inch. 3 days.	7 days.	28 days.	3 months.
1 of cement	175	255	263	328
1 of { 0·95 cement, 0·05 lime }	143	198	290	333
1 of { 0·90 cement, 0·10 lime }	185	213	304	347
1 of { 0·85 cement, 0·15 lime }	144	222	330	382
1 of { 0·80 cement, 0·20 lime }	110	193	241	309
1 of { 0·70 cement, 0·30 lime }	83	183	200	238

It will be seen from these figures that the substitution of hydrated lime for more than 15 per cent. of the cement effects an appreciable reduction in the strength of the mortar. The results are not concordant, but, on the whole, the use of smaller percentages seems to be, if anything, beneficial to the strength after 28 days. The addition of hydrated lime made the mortar work "fatter," or freer.

We do not appear to have any comprehensive series of crushing tests on lime-cement mixtures, but it is to be expected that they would, broadly speaking, follow the lines of the tensile tests.

The experiments which we have from time to time carried out in connection with the waterproofing of Portland cement concrete have shown that the addition of a small quantity of slaked lime is, undoubtedly, beneficial in this connection. I give you here the result of one test as an illustration.

Amount of water which percolated during 24 hours, under a pressure of 50 lbs. to the square inch, through

a slab made with three parts of sand to one of cement.

First slab	110 c.c.
Second slab	120 ,,
	115

Amount of water which percolated during 24 hours, under a pressure of 50 lbs. to the square inch, through a slab made with three parts of sand and one part of a mixture of 0·97 cement and 0·03 slaked lime.

First slab	4 c.c.
Second slab	10 ,,
	7

(*End of letter.*)

The directors of the Associated Portland Cement Manufacturers were so good as to sanction the issue to the author, free of cost, of sufficient quantities of sand, shingle, cement, burnt gault clay, stock brick, red brick, white lime, and grey lime, for the experiments.

The following is the official description of the material so generously supplied.

Sand.—This was washed pit sand, from the A.P.C.M. property at Northfleet in Kent, all of which passed through a $\frac{1}{4}$-in. mesh.

Shingle.—This was washed pit ballast, from the A.P.C.M. property at Northfleet in Kent, all of which passed through a $\frac{3}{4}$-in. mesh, but was retained on a $\frac{1}{4}$-in. mesh.

Cement.—This left a residue of only 6·8 per cent. on the 180-mesh sieve, and the setting time when tested in accordance with the British Standard Specification was 1 hour 20 minutes initial, and 4 hours final. The tensile strength at 7 days, neat cement, 800 lbs. per square inch ; 3 parts standard sand to one of cement, 395 lbs. per square inch. The expansion under the Le Chatelier test after 24 hours' aeration was only 1 mm.

Burnt Gault Clay.—This was gault clay from the River Medway at Burham, which was burnt, and after-

wards when ground left a residue of about 7 per cent. on the 180-mesh sieve.

Stock Brick.—This when ground left a residue of about 9 per cent. on the 180-mesh sieve.

Red Brick.—This when ground left a residue of about 16 per cent. on the 180-mesh sieve.

Ground White Lime.—This was made on the River Medway from white chalk, and contained over 90 per cent. of caustic lime when freshly ground, and left a residue of 18 per cent. on the 180-mesh sieve.

Ground Grey Lime.—The analysis of this made shortly after grinding was as follows:—

	Per cent.
Silica	6·02
Insoluble residue	0·50
Alumina	2·41
Ferric oxide	0·57
Lime	86·01
Magnesia...	0·65
Sulphuric anhydride	1·12
Carbonic anhydride, Water	2·36
Potash, soda, and loss... ...	0·36
	100·00

This when ground left a residue of 5 per cent. on the 180-mesh sieve.

THE EXPERIMENTS.

Preliminary Note.—It was decided to conduct tests on lime and Portland cement concrete at the age of six months. Metric weights were more readily available in the school. It was found that 1 part by measure of each material weighed:—

	Proportinate weight.
Cement, 760 grammes	1·00
White lime, 390 grammes	0·51
Grey lime, 480 grammes	0·63
Red soorkhee, 570 grammes... ...	0·75
Burnt gault, 490 grammes	0·64
Stock brick, 667 grammes	0·88

Each part filled a glass measure of $\frac{15}{16}$ pint. The materials were first measured carefully, shaken down in the glass measure in precisely the same manner,

and weighed. In all subsequent mixtures the materials mentioned above were used by weight. The sand and ballast were measured, but not weighed. The ballast was screened in the later experiments, as some of it was too large for the size of the cubes. It was used as received, clean washed and uncrushed. The bulk of it passed through a $\frac{3}{4}$-in. sieve. When used unscreened, it was not considered necessary to screen the portion used because so little was of excessive size, i.e. over $\frac{3}{4}$ in. A few stray pieces of larger size seem to have got into the sample after screening on works, a matter of no importance in actual construction.

No. I.

Mixed and moulded : September 22, 1914.
Placed under water : September 24, 1914.
Removed and placed in moist closet : Feb. 24, 1915.
Tested on March 22, 1915.

Composition.—

1 part cement	3 parts sand
1 part white lime	8 parts shingle
1 part red soorkhee	ballast, unscreened

$1\frac{1}{2}$ pints water

Mixed dry, and then with water, *as a whole*. The mixture when placed in moulds might be considered a "dry" mixture. The dry mixing was very thorough so as to get an even colour throughout

TEST No. I.

Specimen No.	Condition.	How Tested.	Results in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side	Shape.		
1	Good	Sides	12·0	14·5	15·0	10 c.	Cube	2,168	1
2	Fair	,,	14·4	15·1	15·8	,,	,,	2,283	2
3	Good	,,	15·0	15·7	15·7	,,	,,	2,269	3
4	Defective	Beds	2·5	10·0	13·5	,,	,,	1,951	4
		Average	...	...	...	...		2,168	4

All the specimens were tested between folds of thick brown paper. No. 2, one corner was slightly bad and honeycombed. No. 4 was a poor specimen to look at, being honeycombed and not solid.

No. II.

Mixed and moulded : September 22, 1914.
Placed under water : September 24, 1914.
Removed : February 24, 1915.
Tested on March 22, 1915.

Composition.—

1 part cement	3 parts sand
1 part grey lime	8 parts shingle
1 part red soorkhee	ballast, unscreened

$1\frac{1}{2}$ pints water

Mortar first mixed dry, then shingle added and mixed, and whole wetted and mixed. A "dry" mixture.

TEST No. II.

Specimen No.	Condition.	How Tested.	Results in Tons. 1st Crack.	2nd Crack.	Failed.	Specimen. Side.	Shape.	Crushing Result, lbs. per sq inch.	Specimen No.
5	Defective	Beds	5·0	9·0	9·4	10 c.	Cube	1,358	5
6	,,	Sides	7·0	8·4	8·9	,,	,,	1,286	6
7	Good	,,	5·5	7·5	13·6	,,	,,	1,965	7
8	,,	,,	6·0	8·0	20·4	,,	,,	2,948	8
	Average	...	...	...	...			1,890	4

All No. II (Nos. 5 and 6 being specially bad) showed signs of unslaked grey lime. 24 hours after moulding it had been noted that the blocks did not present so satisfactory an appearance as No. I, the top surface left by trowel having scaled off. No. 5, not placed quite true in machine, also honeycombed.

No. 6, honeycombed. Nos. 5 and 6, bedded on plaster of Paris, an error of judgment it is thought. Nos. 7 and 8, tested between folds of thick brown paper.

No. III.

Mixed and moulded : September 29, 1914.
Placed under water : October 1, 1914.
Removed : March 3, 1915.
Tested on March 29, 1915.

Composition.—

1 part cement	5 parts sand
3 parts white lime	20 parts shingle ballast, screened through $\frac{3}{4}$-in. sieve
3 parts red soorkhee	

About $1\frac{3}{4}$ pints of water

Mixture worked well into moulds, and seems to be a good practical one.

TEST No. III.

Specimen No.	Condition.	How Tested.	Results in Tons. 1st Crack.	2nd Crack.	Failed.	Specimen. Side.	Shape.	Crushing Result, lbs. per sq. inch.	Specimen No.
9	Fair	Sides	1·5	2·6	5·85	4 in.	Cube	819	9
10	Cracked	,,	—	—	4·40	,,	,,	616	10
11	Better than No. 10	,,	—	—	5·65	10 c.	,,	816	11
12	Better again	Beds	0·9	1·3	7·05	,,	,,	1,019	12
13	As 12	,,	0·9	1·6	5·65	,,	,,	816	13
		Average	...	...	...	...		817	5

Lime would not appear perhaps to have been thoroughly slaked, hence cracks in surface of blocks; which might, however, have formed after removal from water. Tested between folds of thick brown paper.

No. IV.

Mixed and moulded : September 29, 1914.
Placed under water : October 1, 1914.
Removed : March 3, 1915.
Tested on March 29, 1915.

Composition.—

$\frac{5}{7}$ part cement
$1\frac{5}{7}$ parts white lime
$1\frac{1}{2}$ parts red soorkhee
5 parts sand
20 parts screened shingle ballast

A scamped mixture, voids insufficiently filled by the mortar. Intended to show the results of dishonest practice. The correct proportions should have been as in No. III.

TEST No. IV.

Specimen No.	Condition.	How Tested.	Results in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
14	Poor	Beds	1·6	1·7	5·0	10 c.	Cube	723	14
15	,,	,,	0·6	0·8	5·8	,,	,,	838	15
16	,,	,,	0·8	2·4	5·7	,,	,,	824	16
	Average							795	3

All honeycombed and apparently of little strength. Could easily have been broken up by picking. Tested between folds of thick brown paper.

No. V.

Mixed and moulded : September 29, 1914.
Placed under water : October 1, 1914.
Removed : March 3, 1915.
Tested on March 29, 1915.

Composition.—

1 part cement   2 parts sand
4 parts screened shingle ballast
About $1\frac{3}{4}$ pints water

To compare with other tests made.

TEST No. V.

Specimen No.	Condition.	How Tested.	Results in Tons.			Specimen.		Crushing Result. lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
17	Good	Sides	6·5	11·5	21·6	10 c.	Cube	3,121	17
18	Excellent	,,	18·5	21·0	29·1	,,	,,	4,205	18
		Average	...	...	...	...		3,663	2

The blocks were exceedingly good, and gave no trouble at all. Tested between folds of thick brown paper, which, in the case of No. 18, tore across under the strain.

No. VI.

Mixed and moulded: September 29, 1914.
Placed under water: October 1, 1914.
Removed: March 3, 1915.
Tested on March 29, 1915.

Composition.—

1 part cement   3 parts sand
5 parts screened shingle ballast
About $1\frac{3}{4}$ pints water

To compare with other tests made.

TEST No. VI.

Specimen No.	Condition.	How Tested.	Results in Tons.			Specimen.		Crushing Result, lbs. per. sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
19	Good	Sides	15·5	18·5	22·4	10 c.	Cube	3,237	19
20	,,	,,	14·5	—	21·0	,,	,,	3,035	20
21	,,	,,	19·5	—	21·3	,,	,,	3,078	21
Average								3,117	3

All blocks very slightly honeycombed. It was not possible to observe second cracks in Nos. 20 and 21 as they appeared to crack and fail simultaneously. No. 20 not so good a block as Nos. 19 and 21.

No. VII.

Mixed and moulded : October 13, 1914.
Placed under water : October 15, 1914.
Removed : March 18, 1915.
Tested on April 13, 1915.

Composition.—

1 part cement 6 parts sand
1 part white lime
1 part red soorkhee

Mixed all together as a mortar test with a slight accidental excess of water, and packed into moulds as quickly as possible.

TEST No. VII.

Specimen No.	Condition.	How Tested.	Result in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
22	Poor	Beds	—	—	3·2	4″	Cube	462	22
23	Good	,,	0·5	1·2	4·6	10 c.	,,	665	23
24	Poor	,,	0·2	0·6	4·8	10 c.	,,	717	24
25	Good	,,	0·8	—	5·4	4″	,,	756	25
26	,,	Sides	—	—	4·7	10 c.	,,	679	26
27	Poor	,,	—	—	4·6	10 c.	,,	710	27
28	Good	Ends	—	—	1·9	3″×3″×4½″	Prism	473	28
29	Fair	Beds	—	—	2·4	50 sq. c.	Cube	694	29
Average								645	8

No. 22, damaged corner and honeycombed bottom or bed, area reduced by ½ sq. in. to 15½ sq. in. No. 24, damaged corners, area reduced by ½ sq. in. to 15 sq. in. No. 27, damaged corner, area reduced by 1 sq. in. to 14½ sq. in. No. 29, corners slightly rounded, no reduction made in area for this. All specimens tested with four thicknesses of brown paper top and bottom, as in every other experiment.

No. VIII.

Mixed and moulded: October 13, 1914.
Placed under water: October 15, 1914.
Removed: March 18, 1915.
Tested on April 13, 1915.

Composition.—

1 part cement 6 parts sand
1 part grey lime
1 part red soorkhee

Mixed all together as a mortar test in the same way as No. VII, with a trifle less of water.

TEST No. VIII.

Specimen No.	Condition.	How Tested.	Result in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
30	Fair	Sides	—	—	3·7	4″	Cube	518	30
31	,,	Beds	—	—	3·55	4″	,,	497	31
32	Good	,,	—	—	3·4	4″	,,	476	32
33	Fair	,,	—	—	3·35	4″	,,	469	33
34	Good	,,	—	—	3·70	10 c.	,,	535	34
35	Fair	,,	—	—	3·90	4″	,,	546	35
36	Defective	Ends	—	—	1·25	3″ × 3″ × 4½″	Prism	311	36
37	,,	Sides	—	—	1·90	10 sq. c.	Cube	549	37
	Average	...	...	...	...			488	8

No. 30, edges and corners frayed. No reduction of area made in calculation. No. 33, expansion cracks. No. 35, same as No. 30. No. 36, defective corners and edges, bottom or bed was honeycombed and frayed. No reduction made on area. No. 37, same as No. 30. All specimens tested with four thicknesses of brown paper top and bottom.

No. IX.

Mixed and moulded: October 13, 1914.
Placed under water: October 15, 1914.
Removed: March 18, 1915.
Tested on April 13, 1915.

Composition.—

I part cement and 3 parts sand
A dry mixture

Mixed as a mortar to compare with Nos. VII and VIII.

TEST No. IX.

Specimen No.	Condition.	How Tested.	Result in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
38	Fair	Beds	14·0	—	20·7	10 c.	Cube	2,991	38
39	Good	,,	11·0	—	19·7	4″	,,	2,758	39
40	,,	,,	—	—	20·2	4″	,,	2,828	40
41	,,	Sides	—	—	25·1	10 c.	,,	3,627	41
		Average	...		...	...	... ...	3,051	4

Nos. 40 and 41 were tested without paper as the beds were clean and smooth. Nos. 38 and 39 tested with four thicknesses of brown paper above and below as usual.

No. X.

Mixed and moulded: October 27, 1914.
Placed under water: October 29, 1914.
Removed: March 29, 1915.
Tested on April 27, 1915.

Composition.—

I part cement 6 parts sand
I part white lime
I part burnt gault

Mixed as a mortar in the usual way to compare with Nos. VII, VIII, and IX.

TEST No. X.

Specimen No.	Condition.	How Tested	Result in Tons.			Specimen.		Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.	Side.	Shape.		
42	Good	Beds	2·6	3·4	4·1	10 c.	Cube	592	42
43	Fair	,,	—	—	5·2	,,	,,	751	43
44	Good	,,	2·8	—	6·5	,,	,,	939	44
45	,,	,,	4·3	5·3	6·1	4″	,,	854	45
46	,,	,,	5·5	—	6·9	10 c.	,,	997	46
47	,,	,,	1·9	—	2·6	50 c. area	,,	751	47
48	,,	Ends	2·7	—	2·9	3″×3″×4½″	Prism	650	48
49	,,	Beds	—	—	6·7	4″	Cube	938	49
			Average	...	...	...	...	809	8

No. 43 slightly honeycombed. All bedded as usual between brown paper.

No. XI.

Mixed and moulded: October 27, 1914.
Placed under water: October 29, 1914.
Removed: March 29, 1915.
Tested on April 27, 1915.

Composition.—

1 part cement — 6 parts sand
1 part white lime
1 part stock brick

Mixed as a mortar in the usual way to compare with Nos. VII to X inclusive.

TEST No. XI.

Specimen No.	Condition.	How Tested.	Result in Tons. 1st Crack.	2nd Crack.	Failed.	Specimen. Side.	Shape.	Crushing Result, lbs. per sq. inch.	Specimen No.
50	Fair	Beds	4·6	—	4·9	4″	Cube	686	50
51	,,	,,	2·7	—	4·0	4″	,,	560	51
52	Good	,,	4·1	—	5·85	4″	,,	819	52
53	Fair	,,	2·9	—	4·6	10 c.	,,	665	53
54	Good	,,	—	—	4·7	4″	,,	658	54
55	,,	,,	—	—	4·7	4″	,,	658	55
56	,,	,,	—	—	2·4	50 sq. c.	,,	694	56
57	,,	Ends	—	—	2·25	3″×3″×4½″	Prism	560	57
		Average	...	...	...	...		663	8

Nos. 50, 51, and 53 corners frayed. No reduction made. All tested between brown paper as usual.

No. XII.

Mixed and moulded: October 27, 1914.

Placed under water when still in moulds: October 29, 1914.

Removed from moulds: November 17, 1914.

Removed from water: March 29, 1915.

Tested on April 27, 1915.

Composition.—

2 parts white lime 6 parts sand
2 parts red soorkhee

Mixed as a mortar, in the usual way, to compare with Nos. VII to XI inclusive.

All broken pieces of 4-in. cubes. Nos. 58, 59, and 60 roughly triangular in section, 4 in. high. Nos. 61 and 62 irregular bits of triangles, about 4 in.

high each, and very rough, and No. 61 hardly a fair test.

In addition to the above an attempt was made to test for tensile strength 5 or 6 briquettes, all but 2 breaking in placing into machine. Results of 35 lbs. and 30 lbs. to the square inch were obtained on the other two. The sections were 1 in. by 1 in., and the briquettes were imperfect and frayed. It may, however, be fairly assumed that the tensile strength of No. XII is from 15 lbs. to 30 lbs. per square inch, say an average of $22\frac{1}{2}$ lbs. It seems fair to assume that, with well-made regular cubes, the crushing ultimate strength of the same material would have been about 225 lbs. to the square inch. This strength is assumed in the analysis.

TEST No. XII.

Specimen No.	Condition.	How Tested.	Result in Tons.			Specimen, Shape, Size, etc.	Crushing Result, lbs. per sq. inch.	Specimen No.
			1st Crack.	2nd Crack.	Failed.			
58	Rough	Ends	—	—	0·2	7 sq. in., 4 in. deep prism	64	58
59	,,	,,	—	—	0·3	$5\frac{5}{8}$ sq. in., 4 in. deep prism	119	59
60	,,	,,	—	—	0·3	6 sq. in., 4 in. deep prism	112	60
61	Bad	,,	—	—	0·15	8 sq. in., 4 in. deep prism	42	61
62	,,	,,	—	—	0·3	8 sq. in., 4 in. deep prism	84	62
Average						... ...	84	5

No. XII proved an extraordinary kind of experiment. The mortar hardened so slowly that the moulds had to remain under water unopened till November 17, 1914. When opened, a great many cubes and briquettes broke, the mortar adhering more strongly to the moulds, though well oiled, than to itself. Indian masons are known to mix oil and lime with soorkhee to make a kind of putty, and they employ raw sugar dissolved in water to mix mortar with, to

improve the lime. It is possible the oil on the moulds combined with the mortar to make a stronger joining. One briquette came out perfectly, and broke by its own weight when handled. It was so very obvious that the white lime, of itself, had no setting power under water or in damp air, that no separate tests of lime and sand were considered necessary.

In repeating the experiment it is suggested that the largest size of cubes the machine available will take be moulded, in wooden boxes lined with oiled paper, placed at once under water, and left there as long as desired undisturbed, moulds and all. The moulds should be iron clamped to weigh more than water.

PART III. CONCLUSIONS.

Concrete Tests.

It will be noticed that two sets of tests, Nos. I and II, were made with béton (1 : 2 : 4 equivalent) ; one set, No. III, with béton (1 : 3 : 5) ; and one set, No. IV, with a scamped mixture supposed to be béton (1 : 3 : 5). To compare this béton with Portland cement concrete, two more tests were made, No. V (1 : 2 : 4) and No. VI (1 : 3: 5).

ANALYSIS OF CONCRETE TESTS.

Test No.	No. of Samples.	Proportions.								Average Strength in lbs. per sq. inch at 6 months.
		Cement.	Remainder.	Cementing Material.	Remainder.	Mortar, less Sand.	Sand and Shingle.	Mortar.	Coarse Material.	
I	4	1	13	2	12	3	11	6	8	2,168
II	4	1	13	2	12	3	11	6	8	1,890
III	5	1	31	4	28	7	25	12	20	817
IV	3	1	40	4	37	6	35	13	28	795
V	2	1	6	1	6	1	6	3	4	3,603
VI	3	1	8	1	8	1	8	4	5	3,117

Mortar Tests.

No. VII.—1 cement, 1 white lime, 1 red sorkhee, 6 sand.

No. VIII.—1 cement, 1 grey lime, 1 red soorkhee, 6 sand.

No. X.—1 cement, 1 white lime, 1 burnt gault, 6 sand.

No. XI.—1 cement, 1 white lime, 1 stock brick, 6 sand.

No. XII.—No cement, 2 white lime, 2 red soorkhee, 3 sand.

No. IX.—1 cement, no lime, no soorkhee, 3 sand.

The idea was to test 1 : 3 mortars of all kinds as far as possible. It will be noticed that the chief difficulty was the classification of soorkhee. Is it part of the sand, or part of the mortar, or should it be left out of consideration?

ANALYSIS OF MORTAR TESTS.

Test No.	No. of Samples.	Proportions.								Average Strength in lbs. per sq. inch at 6 months.
		A			B					
		Cement.	White Lime.	Grey Lime.	Red Soorkhee.	Burnt Gault.	Stock Brick.	Sand.	Proportion. A : B	
VII	8	1	1	—	1	—	—	6	2 : 7	645
VIII	8	1	—	1	1	—	—	6	2 : 7	488
X	8	1	1	—	—	1	—	6	2 : 7	809
XI	8	1	1	—	—	—	1	6	2 : 7	663
XII	5	—	2	--	2	—	—	6	1 : 4	225*
IX	4	1	—	—	—	—	—	3	1 : 3	3,051

Remarks on Concrete Tests.

Unfortunately, there was no time to make tests of lime, sand, and soorkhee concrete. The mixture em-

* Assumed strength if experiment had been successful.

ployed would have been 1 lime, 1 soorkhee, 1 sand, and probably 6 of shingle, and the results the writer would have anticipated should have been about 300 to 500 lbs. to the square inch, after six months. Combining the mean of these, say 400 lbs., and the average result of No. V, we get a new average of about 2,000 lbs. to the square inch, which agrees pretty well with the average result of the two tests Nos. I and II. It seems to the writer that if an extended series of tests were made, covering the range of 1 : 2 : 4 and 1 : 3 : 5 variations of mixtures of cement and lime mortars from cement mortar at one end to lime mortar at the other, it would be easy to plot a curve, which might approximate in the case of concrete to a straight line, to cover every possible variation with sufficient accuracy for all practical purposes. If carefully carried out, these results would undeniably encourage the employment of cement to improve lime concrete at comparatively small cost. Cement has been so employed in India since Portland cement was first manufactured.

Remarks on Mortar Tests.

It will be noticed that the cement and sand test beats all other tests hollow. The writer does not, however, consider that the question has been settled by these experiments. In the first place the red soorkhee, burnt gault, and stock brick, supplied, were all well burnt, and Indian traditional practice demands a certain proportion of half-burnt material. As a matter of fact, the purer the lime the more under-burnt the proportion of soorkhee necessary to make it set under water. With the well-burnt material supplied, the white lime had insufficient inducement to set under water, and the writer is a little surprised it did so well. Cement seems, besides, to have a kind of chemical (?) affinity for lime, and helps it to set. The writer has used cement, lime, and sand without soorkhee in work set in air, but has always used soorkhee in underwater and underground work, because he could not make the necessary experiments to decide whether it could safely be left out.

Many more experiments are required before it can

be stated with any certainty what effect soorkhee has on English limes.

As a matter of fact, civil engineers in India differ in opinion regarding the extent to which soorkhee should be burnt. Brick-earth in India often contains salts and the presence of such salts in mortar leads to decay. If the brick-earth, however, is thoroughly well burned, short of vitrification, the salts appear to lose their power for evil. Hence the existence of two opinions, the one holding the possibility of salts existing in the soorkhee to be a greater drawback than hindrance by over-burning to setting. The writer himself thinks that every kind of lime requires its own kind and quantity of soorkhee to obtain the best results. Since under-burnt soorkhee has not the crushing strength of good sand, it is obvious that to allow an excess of the weaker material is bad engineering. It is possible, too, that well-burnt soorkhee is just as good as under-burnt soorkhee, since Indian opinion is practically evenly divided on this point. Further, we have no scientific knowledge of the action of Portland cement on lime without soorkhee to assist setting under water. We have no exact knowledge how cement, lime, and soorkhee combine together, or how far the combined bulk goes to fill interstices in standard sand, which is just as important as designing mortar to fill the interstices in aggregate.

Mixtures of Cement and Lime in India

are not recent, probably as old as cement manufacture itself, necessity being the mother of invention. The writer consulted Mr. William Harvey, a retired M.Inst.C.E., who was employed on railway work in India for many years, and who was for ten years previous to retirement in 1904 chief engineer of the Bombay, Baroda, and Central India Railway. Mr. Harvey, while engaged from 1871 to 1875 on the construction of the Ravi bridge, etc., of the Panjab Northern Railway (now part of the North Western Railway), employed Portland cements mixed with the "ghooting" lime locally obtained, in varying proportions, in order to expedite setting of the work. To use cement alone would have been too expensive.

Without the cement, work would at times have been unduly delayed, especially the sinking of the wells used in the foundations of the piers. At times, it was imperative that the sinking of the wells should proceed as soon as possible after a cylinder of brickwork had been built and "loaded"; this could not be done till the brickwork had set, and so, to avoid delay, the addition of Portland cement to the lime was most valuable. Large numbers of concrete blocks from 2 to 27 cub. ft. were used in the river training works, and in protecting the piers from scour, and these frequently would not have been ready in time but for the quick setting properties of mixtures of cement and lime mortars, as compared with limes alone.

Mr. Harvey unfortunately kept no notes of the exact mixtures he used, but he is confident that the use of Portland cement mixed with the local "ghooting" limes was satisfactory and economical. He is also prepared to defend the use of mixtures of "fat" with hydraulic natural limes; and the mixture of either or both with Portland cement where it is advisable to delay the setting of the latter.

Comparisons of Results

between such mixtures and cement mixtures are difficult on account of this very property of quick setting. Lime and sand mortars do not set thoroughly hard for centuries. Cement and sand probably attain maximum strength in a few years. The results given in this paper show tests at six months. As a matter of fact a fair comparative test for the béton mixtures would have been at six to ten years, to the six months for Portland cement concrete.

Mixtures of Cement and Lime in this Country

are not so rare as might be believed. The writer knows of at least one builder who employs the mixture, and he also knows of several instances of pointing and walling in private houses, in which a mixture of $\frac{1}{4}$ ton cement to every cubic yard of lime was used with

astonishingly good results at a few months, with the advantage that, although in one case the work was done some ten years ago, the pointing appears to be still hardening. The proportion of cement to lime was about 1 : 4.

The Question of Cost

is, of course, an important one. The cost of labour in mixing, handling, and applying such mixtures, would, with a little experience, be probably the same. If the slower setting does not affect the cost, or other considerations, it is no disadvantage. Mixing lime with cement gives a fatter mortar, easier to work. In a country where cement is nearly as cheap as lime, there seems to be no object in using a mixture, unless it is to get a fatter and possibly more watertight mortar. Where lime is cheaper than cement, it is obvious that in the course of time a mixture of, say, 1 cement, 1 lime, 1 soorkhee, and 6 sand, may be better and more reliable than 1 cement to 4 sand. In the former case we get 2 of cementing material to 7 of other material, that is, 1 : $3\frac{1}{2}$; in the latter case 1 : 4. The soorkhee, too, being as finely ground as the cement and lime aids to fill the interstices of the sand. Clay is recommended for adding to the water-tightness of cement concrete. It is a question whether soorkhee would not be better.

In the East, where carriage is very heavy, where lime is cheap and good, and the native masons understand its use better than the use of cement, such mixtures have a very high value. In Rangoon, a large seaport, the cost of cement to lime in 1909 (see Part I) was as 20 : 3. In the Shan States, or similar localities in Burma, where the cement had to be carried many miles by pack-animal or man-carriage, the differences were enormous, probably as 20 : 1 to 100 : 1, in the same year. In January 1909, *Indian Engineering* gave the price of lime in Dehra Doon in Northern India, at site of kiln, as Rs. 60 per 100 maunds. A maund is roughly 80 lbs., and a rupee 1s. 4d., so that the rate works out to 1s. per 100 lbs., about £1 2s. 6d. per ton of 2,240 lbs.

These conditions prevail everywhere in rough, undeveloped lands, and it is of the highest importance to the structural engineer and to capital that cement can be utilized as described in these pages. In the absence of reliable tests we are all working in the dark, by rule of thumb.

DISCUSSION.

MR. D. B. BUTLER, Assoc.M.Inst.C.E., F.C.S., M.C.I. :—There is an immense amount of experiment here which, as the author himself suggests, might be carried a great deal farther. The use of soorkhee with lime seems to me to be somewhat on a par with that of the volcanic material known as pozzolana, which is something, I believe, of the same composition chemically as soorkhee. It is really a sort of half-baked clay, more or less burnt, and is used, I believe, largely at the present time in Italy and elsewhere with lime for work under water. As a matter of fact, I believe something similar was used by Smeaton in building the Eddystone Lighthouse. He used hydraulic lime with a kind of pozzolana, with first-class results. That was really before Portland cement was known as a constructive material. Reverting to the paper itself, I think possibly one reason why the lime gives such good results is to be found on page 232, where the analysis shows it to contain as much as 6·02 per cent. of soluble silica; this soluble silica would combine with the other constituents of the mortar, and the chemical combination thus formed probably accounts for the strength of the material.

The whole paper is so full of matter, that without time to look thoroughly into it it is impossible to criticize it with any success. A very interesting part of the letter from the Associated Portland Cement Manufacturers is that dealing with the effect of the addition of lime on the water-penetration-resisting qualities of the mortar. I think it is on page 231, where a slab made of 3 to 1 mortar, without slaked lime, allowed an average percolation of 115 c.c., whereas with 0·03 per cent. of slaked lime added, the percolation only averaged 7 c.c. I think, that that is largely due to the closing of the pores.

The lime practically closed the pores of the mortar; another reason is, that the addition of the lime, as mentioned in the paper, makes the mortar work fatter, and thus produces a more dense and less porous material. I think, that that is one reason why the results given with the cement and lime are as good as they are. On page 249, in the comparison of results, the author says: "Lime and sand mortars do not set thoroughly hard for centuries. Cement and sand probably attain maximum strength in a few years." I think there are many instances where a material, which attains its greatest strength in three years, is preferable to one, which takes fifty or a hundred years to attain its greatest strength; that fact must be carefully borne in mind in using lime as against Portland cement.

MR. MORGAN E. YEATMAN, M.A., M.Inst.C.E., etc., next spoke.

MR. A. R. SAGE, M.C.I.:—I should like to thank the author for his interesting paper, which has appealed to me particularly, as I was present at some of the tests. I cannot understand the results of tests Nos. 1 and 2. It seems as though Experiment No. 4 of Test No. 1, page 233, is possibly the correct value, or else the results in Test No. 2 are too low, because I think it is beyond dispute, that grey stone lime is stronger than whitewash, and we cannot call ordinary chalk lime and water much else. Grey stone lime, at any rate, is stronger than ordinary chalk lime.

On the bottom of page 235 there is a note referring to the slaking of the lime. It might be of interest to confirm that. The lime was not slaked. It was slaking, and it is *still* slaking. This is obvious from the fact, that the casks, which once held it are able to hold it no longer, and the volume of the remaining lime is half as much again as that of the casks.

The tests with cement do not seem to prove anything. I cannot see, that there is any very useful purpose served in comparing these tests with cement, and I think, a much more valuable series could be carried out on the following lines:—1*st series:* chalk lime concrete—chalk lime, sand, and aggregate (if such a thing be possible); 2*nd series:* chalk lime

and soorkhee concrete—chalk lime, soorkhee, sand, and aggregate. We could thus see the effect of soorkhee. *3rd series:* stone lime concrete—stone lime, sand, and aggregate, compared with a *4th series:* stone lime and soorkhee concrete—stone lime, soorkhee, sand, and aggregate—again to see the effect of the soorkhee.

The adding of cement to the lime concrete is obviously to hold it together, while the lime has a chance to harden, but if soorkhee is to be added it would be interesting to find out, whether it effects an improvement, whether it is an inert material, or whether it gives some definite result. There certainly appears to be something in it from the tests. I was sceptical as to the results, when I saw the tests in progress, and I did not think the cubes would ever be removed intact from the moulds. As a matter of fact, some fell to pieces because of the weak nature of the mixture. I was very much surprised to see, that the majority of the blocks held together, and many of the results of the crushing experiments exceeded the best, I had anticipated.

With regard to what actually takes place in the setting of these mixtures, I suggest that, owing to the finely divided state of the soorkhee and the lime, some chemical action, similar in character to that which takes place in the Portland cement kiln, may occur, because the soorkhee is possibly as fine as some of the best Portland cement. At any rate, it seems to me to be worth while to carry the tests farther, both from a physical and a chemical point of view; but this, of course, needs money, and I think the people to approach for that are those, who are interested in the use of soorkhee. Is there any objection to, or difficulty in the way of the manufacture of Portland cement in India? If it cannot be manufactured there, it is worth while going on with the subject of soorkhee; but if it can be manufactured in India I think, the question of soorkhee does not count for very much. I do not think we can admit the remark Mr. Phillips made with regard to brickwork and reinforced concrete. There is no question that brickwork, well bonded, is very good, but the strength of brickwork depends very largely upon the bond for one thing and upon the strength

of the mortar for the other. If good Portland cement mortar be used for brickwork, having the joints large enough and giving it reasonable time to set, the chances are that the majority of bricks would break before the joint. Therefore the brickwork can hardly be said to reinforce the concrete. On page 247 there is a reference to half-burnt brick. I can understand the possibility of an increase in strength with the use of thoroughly burnt brick, which, after all, is of a similar nature to the clay used for Portland cement burning. The strength of Portland cement depends on the chemical combination of the constituents by heat; half-burnt brick is really inert material. With regard to the waterproofing properties of cement concrete being increased by the addition of lime, the effect is probably similar to that obtained by punning concrete—viz. a denser mixture.

MR. W. G. PERKINS, M.C.I. (District Surveyor for Holborn) :—The author has begun his paper with a discussion of the manner, in which mortar is prepared in India, and the point that particularly interests me is the introduction of soorkhee in the composition of mortar. That is done, I presume, in order to give hydraulic properties to the fat limes employed. I should be glad if Mr. Phillips will tell us something about the chemical composition of soorkhee. The use of this material leads me to speak of the practice, that obtains in the neighbourhood of London, where mortar is frequently compounded with red bricks obtained from the demolition of houses from one hundred to two hundred years old. These houses are pulled down and the speculating builder uses the old bricks by grinding them into a mortar, composed of one part of grey lime to three parts of broken brick. As a rule, provided that this lime is properly slaked —and it *must* be properly slaked—you will obtain a very good mortar. I have here some samples of the kind of brick, to which I refer. I propose to hand these round, together with a few samples of mortar, made with such brick taken from various buildings in my district. The next thing I want to hand round is a sample of plastering, that was being applied to the walls of a building on May 11th. This plastering

was taken from a wall and kept in my office until now. It is made of grey stone lime run into putty, probably three weeks prior to use. Two bushels of putty were put into a pan and the pan filled up with brick of the character, I have shown you. In this short time you will see, that the plastering, which is really only mortar, with a certain amount of hair in it, has set very well ; as time goes on it will no doubt get very much harder. Then I would like to hand round another sample of mortar, made from the same kind of brick and blue lias lime, one part of lime to three of the brick. Then another sample, this time made from Portland cement and clinker, obtained from a gasworks, 1 of cement to 4 of clinker. Here is a piece of the brickwork, built with this cement. The bricks were wetted before being laid. You will notice how extremely well the mortar made with cement and clinker has adhered to the Fletton brick. It is generally assumed that cement does not adhere well to this class of brick. These are not special examples. In all cases the buildings, constructed with these materials are buildings, erected by builders as a speculation. They therefore only just comply with the by-laws—that is to say, the mortar is compounded only precisely as the by-law directs ; there is no extra lime or cement in it. With regard to the use of lime and Portland cement mixed together, I think, that is very good practice. A number of large blocks of artisans' dwellings have been erected under my supervision, and the mortar used was 1 of Portland cement, 1 of grey stone lime, and 7 of broken brick and sand. This has set remarkably hard, and whenever they want to make an alteration, involving the removal of brickwork, the bricklayers have quite a job to cut through it. Such a mortar is really gauged mortar.

Mr. H. KEMPTON DYSON (Secretary C.I.) :—I think I might just explain one or two views, that I had in helping with these tests. Firstly, if you look on page 245, you will find, that Mr. Phillips has divided the tests into two parts, the first concrete and the second mortar. Well, the first set was more or less based upon Mr. Phillips' practice in India,

just to tell the practical strength of concrete mixture in which soorkhee was added to lime, and cement as well. Those tests are purely of a practical character, and they do not go, of course, so far as one would wish. There is room for plenty more experimenting, but the first test certainly did surprise me as to the strength, that could be obtained, where only a small proportion of cement was mixed with the lime and the other materials. The other set of mortar tests I think I was rather responsible for suggesting, because the object of that was to compare the materials, so far as one could, with only a small number of experiments. You will see that in Tests Nos. 7 and 8, for instance, the variable is the kind of lime. Notwithstanding what Mr. Sage suggests about the grey lime, those two tests on the mortar, where the only variable is the kind of lime, infer that grey lime is inferior to the white lime as we used it. The only suggestion I can make as to the cause is, that the grey lime was not slaked, or not properly slaked, because one found that the top surface of the blocks in the moulds had burst upwards, and so evidently the blocks had expanded, and they probably went on expanding afterwards. Experiments Nos. 10 and 11 should be compared with No. 8, the variables there being in the kind of soorkhee. One sees from these experiments, that burnt gault appears to be a better material than stock brick, and that stock brick in turn is a better material than red brick for making the soorkhee. Of course, these are only a small number of experiments, and one cannot assert anything very definite from them. Regarding Experiment No. 12, white lime is used there with red soorkhee and sand alone without cement, and there is a very poor result from it ; but one wonders why you have such a poor result there and such a high result in the concrete in Test No. 1, but I think, one wants to use a bit of judgment in criticizing the comparison between those two. For one thing the Portland cement makes a great deal of difference, and for another thing the proportions in Experiment No. 12 as between lime, which might be called the cementing medium, and the sand, is as 1 to 3, and between lime and sand and soorkhee is as 1 to 4 ; in such

a mixture the voids are not filled. The object of testing sand and cement in the proportions of 1 to 3 is merely to gauge the cementitious value of the cement—that is to say, if we fill up the whole of the voids in a 1 to 2 mixture, as is customary, we do not really gain a full measure of the increase in strength with age of the cement, and we do not really measure its full cementitious value. You want to use cement in sufficient quantity to fill the voids, and then you can directly measure from comparative experiments the value of one cement against another and its value with increase of age. Therefore in Experiment No. 12 the voids were not perhaps completely filled. It must be remembered that those proportions are by volume and not by weight. Bearing upon that, on page 230 there is a remark about the waterproofness of lime and cement mortar. I take it, that the proportion there mentioned is not by volume, but by weight, and if that is so the 0·03 per cent. of lime that was added, it should be remembered, is more bulky than 0·03 per cent. of cement would be, and it is the greater bulk, I suggest, that helped to fill the voids in that slab and so closed the pores against the percolation of water. Now, I believe that one of the elements in the strength of the mixture in No. 1 was that the voids were very completely filled. If you compare the tests on various mixtures of concrete, you will find, that the first essential in getting a good result from concrete is, that you must fill the voids. You will get more value from filling the voids than from anything else. Density is the first essential in making good concrete, and some people have been misled by that fact into saying, that the addition of clay was an advantage in making mortar. Well, if it is a mixture of 1 to 3, the voids are not filled by the cement, and then it is an advantage to fill those voids up, even if you only do it with clay. Get density first of all. As soon as you get a mixture of 1 to 2 the voids are completely filled by the cement, and then you find, that the clay reduces the strength and can be looked upon as dirt, though it is better to have dirt than a lot of voids. In the mixture No. 1 undoubtedly the voids were very completely filled, perhaps overfilled,

and one might have got a greater strength, if one had not overfilled the voids ; but one great element in obtaining that high strength is, I suggest, that the voids were completely filled, whereas the very low test No. 12 is a case where the voids were not filled. In some of the weaker mixtures the voids are not so completely filled as in No. 1, and the result is that strength drops rapidly. Take Nos. 3 and 4, for instance. There there are such poor proportions, that the voids are not filled, and that is the real reason, I think, for such a marked reduction in strength. With a poor concrete, if you put in lime and soorkhee, it will help to fill the voids and so give density to start with. Of course, the mixture of cement with lime has often been used by workmen—bricklayers—in this country. I believe, that one of the workman's ideas in mixing lime with cement mortar is to make it work fatter, but it certainly sets well, and these tests, at any rate, suggest that there is some marked advantage, as regards strength, to be derived from such a mixture, while Mr. Phillips assures us of the economy of it.

MR. HENRY J. HARDING, M.C.I. :—My experience has been practically confined to blue lias lime. I have used blue lias lime with cement with sundry and great advantages, and I have been through a good many experiments with blue lias lime. Perhaps my most interesting experience was my association with that grand engineer Mr. Greathead (who died before his "tube" fame became generally appreciated), and the late Mr. George Deacon. As regards the latter, this must have been about twenty or thirty years ago, at the time when the Liverpool waterworks were being made. My connection with him at first was confined to furnishing the lime for the experiments on that occasion ; but I believe a large quantity of blue lias lime was used in the construction of the dam of the Vyrnwy reservoir. Later on I had another experience, when they made the Kendal waterworks, and I believe blue lias lime was used again there. The principal experiment, that I remember, was made here in London, and on behalf of my principals (Messrs. Greaves, Bull & Lakin), I furnished

the lime, and also went with Mr. Fred King (the contractor, for whom I was experimenting in slaking by steam) and saw how he (Mr. Deacon) made his experiments. The principal thing in all lime, whether blue lias or grey lime, is that it must be perfectly slaked, or, rather, perfectly divided, or else you will never get the best result from your lime. Mr. George Deacon's practice was this. He had a trough, and broke the blue lias lime up in small pieces about the size of walnuts and slaked it with water carefully in the trough, and then he had a machine made something after the style of a paint-mixer. It had five or six long rollers. After he had got the lime into a powdered state, he reduced it into something of a slurry, and he passed it through this roller mill, and from that he got his cementing ingredient for experiment. The result of witnessing that experiment was that I saw, that it was not commercial here in the South of England. The cost of producing that result was considerably more than the then cost of Portland cement ; but I thought to myself, " Well, is there no other way to make that perfect division ? " I tried further experiments, and found, that if you got your lime and slaked it, and then passed it through a pestle and mortar in a dry state, so as to ensure a perfect division, it did not matter about its being wet. But Mr. Deacon's chief point was, that after it was once moist he never let the lime get dry. If it was not used the same day the lime was to be put into a trench and covered up and kept moist. That was simply going back to the old Roman system of " souring " the lime for months. Now, this of course increased the cost enormously. I then tried to see, whether it could not be done in a dry, powdered state, and found that if you powdered this lime to its finest division and immediately perfectly dried it, it was possible to keep lime in that state for months and even years, provided the lime was prevented from extracting moisture from atmosphere, and I actually patented in England and Germany a rough idea of a method to produce same. But the Portland cement had gone ahead then, and none of the Portland cement companies would take my idea up, and situated as I then was as an agent I could not go on with the

patent, and after paying the English fees for eight years I dropped it ; but there is the patent, although I have forgotten the number and exact date.* Roughly, the patent was to slake the lime with hot water or steam, and then pass it into a hot grinder with hot air and blow the divided lime up a shaft or cylinder, so that, as soon as it got over the top, it would fall of its own gravity into bins prepared for the purpose. Anything that was too heavy to reach the top of shaft would fall down again and go through a further process of slaking or grinding. You will remember, hydraulic lime does not slake all at once, 95 per cent. might slake in five or ten minutes, and the other 5 per cent. might take hours to slake. It all depends on the burning and quantity of water applied ; further, it is more difficult to slake blue lias lime in its ground state than in its lump or unground state. However, that is my experience as regards blue lias lime. As I have told you, I dropped the patent, and since that time my experience has gone more in the direction of aggregates, which I consider of importance to a good concrete, equal to, if not greater than lime. Mr. Deacon carried out the Kendal waterworks with blue lias lime in place of Portland cement, and in face of great opposition on the part of not only the other engineers but of the Council, I believe, but I am speaking from memory of some twenty years ago.

Mr. ALLAN GRAHAM, A.R.I.B.A. :—There is no doubt, the paper is a very original one, the author having developed lines of research, to which we are not accustomed. To our Colonies, where Portland cement is difficult to get, these experiments will be of considerable value, and it certainly is an eye-opener to see, how the addition of a little lime to the concrete, even with so much aggregate added, seems to reduce the concrete so little, as compared with the pure Portland cement concrete. For that reason I think, there is something in this line of research, only of course in our country this kind of research does not receive the official recognition or encouragement it should, and which it receives in Germany. We are too imperially

* It is No. 3203, February 8, 1898.

minded to pay attention to small matters of that sort, so I think that as far as India is concerned if, as Mr. Sage suggested, we cannot make Portland cement works in the neighbourhood, we should assist them in doing something to get really good concrete. From what the author of the paper says, there seem to be no Indian experiments, upon which he could lay his hands. I might in this matter be like Oliver Twist and ask for more. As far as I can see, there has been a good deal said about the adding of clay to concrete. I think this is the recrudescence of an old heresy. When Portland cement was a different material from what it is to-day, even some American authorities of ten years ago used to advise the addition of clay. Of course, our English chemists, who had a good cement much earlier than the Americans, proved that the addition of clay in the case of American cement was really of value, inasmuch as there was so much free lime in the Portland cement that the clay was necessary to kill it, so I am afraid the addition of clay is a recrudescence of that old theory.

MR. A. STEWART BUCKLE, M.S.E., M.C.I. :—I have done a good deal of work in India in the same way as Mr. Phillips describes. I have even worked in the same country, in Burma. Earlier in Assam I had experience of the effect of the great earthquake of 1897 on railway construction. Some bridges, especially those that were on wells, disappeared, but not all by any means. That was simply because of the foundations going completely, but it was wonderful how some of the work stood. In the case of one big bridge, with three spans of 100 ft. and eighteen of 60 ft. and about 60 ft. high, a few of the piers broke right off along the mortar joints, but they did not come down, and the trains were run over them, first with light engines, then heavier engines. That was while the rains were on, and it was not known where the cracks were, though it was clear that some piers were out of place. One or two piers were eventually taken down and rebuilt as they had been before—viz. 1 of soorkhee, 1 of lime, and 1 of sand, the regular practice, before

we had this paper to guide us. It was remarkable, that the bricks were often easier to cut away than the mortar. Another case I remember, in which a bridge had to be built in a week. The floods scoured under the lime concrete and soorkhee concrete foundation. I saw the concrete foundation, when the water went down, standing apparently on nothing whatever. The abutment had come forward, and it was only held up by the strength of the ballast wall against the girders. I saw afterwards, that out of about 5-ft. width of concrete there was only about 2-ft. width of clay for it to stand on ; the rest of the concrete was overhanging, and the whole weight of the abutment would have gone but for the girders, the ballast wall being, I suppose, 9 in. or 14 in. thick, no more.

THE CHAIRMAN :—Was that grey or white lime?

MR. BUCKLE :—Grey lime. In Burma some bridge foundations were laid, where there was a regular tidal bore at every full and every new moon. The lime did not set fast enough, so I did what Mr. Phillips recommends, mixed a little cement with the lime, to give time for it to set between the bores, with satisfactory results. One gentleman asked, if cement was not made in India. Well, it is made, but it is not much cheaper than imported cement, and not so good. I think Mr. Phillips will agree with me in that. I should like to say something about my experience in Ceylon with coral lime. I found that soorkhee was not necessary with this lime. To obtain it, all I had to do was to pick it up off the beach, load it into boats, and burn it. It gave just as good results as the soorkhee and lime.

MR. PHILLIPS :—Was that in the air or under water?

MR. BUCKLE :—Not under water, no. My usual practice has been to use cement only when working under water. I have used coal ashes instead of soorkhee in India with as good results. My experience in India has taught me, that after having burnt your bricks the measure of the work is the time, in which you can get your soorkhee ground.

THE CHAIRMAN (MR. C. F. MARSH) said, that with regard to the addition of lime to concrete, he had great faith in the effect of this as increasing the great faith in the effect of this as increasing the watertightness, and had used it on several occasions, adding 5 per cent. of the cement by weight of hydrated lime, which has a considerable effect in reducing the percolation of water. In fact, he considered it as good as, and indeed in some ways better than most of the patent compounds, that are sold. He thought it had very much the same effect as these compounds, since in both cases the general effect was due to the filling up of the interstices. On page 229, with respect to the table giving the tests of the various mixtures of lime and cement, he noticed that the author in reading did not give the 28 days' and 3 months' tests, which he thought really are more interesting than the 3 and 7 day tests. These clearly showed, that up to 0·85 of cement and 0·15 of lime—that is, 17·6 per cent. of lime to the cement—they are really just as good as, or even better, than the cement and sand alone. It would be noticed, that the test, which was carried out by the Associated Portland Cement people, shewed only 3 per cent., and he considered that it would be quite safe to use up to 10 per cent. Some experiments had been carried out in America recently with regard to the adding of hydrated lime to cement for waterproofing purposes. It was found, that up to 10 per cent. there was practically no decrease in strength. He did not quite understand the reference to not slaking the grey lime, as he had always thought, that grey lime had to be slaked before use; but they apparently did not slake it and consequently got some extremely bad results. He thought, that if it had been slaked just before use, they would probably have got very much better results.

MR. HARDING :—I should think, the grey lime would be what they call ground lime, but the difficulty is to find the difference between ground lime and slaked lime.

THE CHAIRMAN :—But I take it that grey lime is the grey stone lime, which is very high in

carbonate; but perhaps Mr. Butler can tell us the percentage.

Mr. BUTLER :—White lime is practically pure carbonate of lime. Grey stone lime is only about 75 to 85 per cent.

THE CHAIRMAN :—And the blue lias lime?

Mr. BUTLER :—Anything from 60 up to 80 per cent.

Mr. HARDING :—Grey lime would be 90 per cent.

Mr. BUTLER :—No, lower than that.

Mr. PERKINS :—I think, grey lime contains about 7 per cent. of clay before burning, and that 7 per cent. of clay would of course give the lime some hydraulic quality.

Mr. BUTLER :—More than 7 per cent. of clay.

THE CHAIRMAN :—At any rate, it is higher than the blue lias lime in carbonate.

Mr. DYSON :—The lime as it came to us was supposed to have been slaked, and it was supplied to us in powder. I understood, it had been ground. This is the powdered white lime in this bottle, and there is another bottle (on the table) of powdered grey lime, but evidently it had not been completely slaked, because it swelled in the mould and subsequently burst the barrels, in which it was supplied.

THE CHAIRMAN :—With regard to Mr. Dyson's remarks as to filling the voids, he makes an important point of Test No. 1 being good, because the voids were filled, but before that he referred to the bad result of the Mortar Test No. 12, because the voids were not filled. It would be observed, that in No. 12 there were 2 parts of white lime, 2 of soorkhee, and 6 of sand, that is to say, 4 to 6. As the soorkhee filled up the voids as well as the lime, the mixture being as 2 is to 3 should be better than the 1 to 3, and ought to give better results, if Mr. Dyson's conclusions were correct.

MR. DYSON :—One of my points was, that the soorkhee was not entirely a cementing medium. In large part I believe it is sand, so that it increases the volume of the sand. I want it to be distinctly understood, that in my view the soorkhee is not entirely effective as a cementing medium. It is partly sand, the coarser parts of it.

THE CHAIRMAN stated that, with regard to clay adding to the watertightness of the concrete, there was a very strong section of opinion in America, that regarded very finely divided clay as a very good material to add, to increase the watertightness of concrete. It was necessary to use great care as to the nature of the clay, and careful tests should be made before using it, but finely divided clay undoubtedly filled up the voids, and although he did not advocate the use of it himself, still, there had been some extraordinary tests on it, which showed that if certain very finely divided clays were used they assisted in waterproofing the concrete. With regard to the use of blue lias lime, Mr. Deacon used the blue lias lime entirely for the reason, that he was building a big masonry dam, and he wished the dam to settle evenly. His fear was, that with the use of cement he would get cracks, because one part of the dam would set hard before the other part had settled down. He used blue lias lime all through his life for masonry dams for the reason that the setting took longer, and therefore the dam could settle itself into place and not be cracked.

MR. PERKINS :—Is there anything in that, do you think?

THE CHAIRMAN :—There is a great deal in it, but I do not think it is necessary.

MR. PERKINS :—That is what I mean.

MR. HARDING :—Will Mr. Phillips say, whether he has had any experience with the white lime used in the south of France by the Marseilles men? I forget the exact name of it. I think, the adoption of the term "white lime" is rather unfortunate, be-

cause this French lime is very white indeed, and it is called white lime there.

THE CHAIRMAN :—Somebody may make a mistake, but I think it is perfectly understood, that the white lime referred to in the paper is chalk lime.

MR. E. A. W. PHILLIPS, in reply to questions, stated that he himself did not recommend the addition of raw clay to cement, but advocated soorkhee, or well-burnt powdered clay, instead. He entirely agreed, that a great many more tests were desirable, extending perhaps over a period of five or six years, or even longer. In testing for the paper the only failures were in No. 12. Every block, excepting in No. 12, was a success ; and in No. 12 the blocks adhered so strongly to the moulds, that they broke during removal, with the result that such tests as were made were carried out upon the most extraordinarily shaped prisms. The tests have all been reported as actually made, and show actual results, good and bad. A strength of 225 lbs. to the square inch is not an excessive assumption, if the blocks had come out properly from the moulds, and agrees with his Indian experience, as he got that strength in a six months' test in India on ordinary white lime, sand, and soorkhee.

INDEX

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THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

(*FOUNDED* 1908. *INCORPORATED* 1909.)

TRANSACTIONS

AND

NOTES

VOLUME VII

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OBJECTS OF THE INSTITUTE.

The objects of the Institute are :—

(a) To advance the knowledge of concrete and reinforced concrete, and other materials employed in structural engineering, and to direct attention to the uses to which these materials can be best applied.

(b) To afford the means of communication between persons engaged in the design, supervision and execution of structural engineering works (excluding all questions connected with wages and trade regulation).

(c) To arrange periodical meetings for the purpose of discussing practical and scientific questions bearing upon the application and use of concrete and reinforced concrete and other materials employed in structural engineering.

The Institute is not responsible for the views of individual authors as expressed in Papers, Letters or Notes, but only for such observations as are formally issued on behalf of the Council.

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CONTENTS

DISCUSSIONS

Note.—For the purposes of War-time economy, reporting of Discussions of Papers read at the Ordinary General Meetings was discontinued, and a mere skeleton of such is given in one or two cases within.

MEMBERS LOST BY DEATH

1915 (not previously reported)

Samuel Douglas Topley, A.R.I.B.A.

1916

2nd Lieut. William Bell, R.E. (Killed in Action, July 26th).

Lieut. J. Hopkinson (Killed in Action).

Captain W. H. Morton Johnson, M.A., F.R.G.S. (Killed in Action, July 2nd).

2nd Lieut. H. Maguire (Died of wounds).

DONATIONS TO LIBRARY, ETC.

We have further to record the following donations, etc., up to the period ending December 31st, 1916 :—

Donor.	*Particulars.*
Adams, Professor Henry	"Strains on braced Iron arches and arched Iron Bridges." By A. S. Heaford.
Batsford, B. T., Mr.	"Reinforced Concrete Regulations," annotated by Ewart S. Andrews.
Chapman and Hall, Messrs.	"Architects' and Builders' Pocket Book." By Frank E. Kidder.
Etchells, E. Fiander, Esqre.	"The dynamic action and ponderosity of Matter." By Waterdale.
	"New principles of Natural Philosophy." By W. L. Jordan.
	"Ocean currents and the System of the World." By W. L. Jordan.
	"Proceedings of the Institution of Mechanical Engineers" (17 vols.).
	"Science Abstracts" (6 vols.).
Faraday Society, The...	"The corrosion of Metals : ferrous and non-ferrous."
Kotasthane, M. M., Mr.	"Reinforced Cement-Concrete Construction." By V. M. Kotasthane.
London County Council, The ...	"Reinforced Concrete Regulations" (2 Copies).
Marsh, Charles F., Esqre.	"Manual of Reinforced Concrete and Concrete block construction." By C. F. Marsh and William Dunn.
Mechanical Engineers, Institution of	"Proceedings," Oct.–Dec., 1915.
Mesnager, A., Mons.	"Cours de Beton Arme." By A. Mesnager.
Sachs, E. O., Esqre.	"Builders' World," "Engineering" and "Public Works" (several vols.).

Donor.	*Particulars.*
Serrailler, L.	Some fifty slides of work done in America in Reinforced Concrete.
University of Illinois, The	"Strength of webs and T-Beams and Girders." By H. E. Moore and W. M. Wilson.
	"The strength and Stiffness of Steel under Biaxial loading." By Professor A. J. Becker.
	"Tests of Reinforced Concrete flat slab structures." By Professors A. N. Talbot and W. A. Slater.

THE CONCRETE INSTITUTE

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SIXTY-FOURTH ORDINARY GENERAL MEETING

WEDNESDAY, NOVEMBER 17, 1915

THE SIXTY-FOURTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held at Denison House, 296, Vauxhall Bridge Road, Westminster, S.W., on Wednesday, November 17, 1915, at 5.30 p.m.

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., M.S.A., F.S.I., F.R.San.I., etc.), in the Chair.

The following elections took place :—

MEMBERS.

STEPHEN MOORSHEAD, Jun., Engineering Contractor, Port Elizabeth, S.A.

GIACINTO SIACCI, Member of the Societá Italiana per lo studio des materiales du Construzione ; Member of the Société des Ingénieurs Civiles et Architects d'Egypt, in Cairo ; Consulting Civil Engineer and Reinforced Concrete Expert, etc., Cairo.

ASSOCIATE-MEMBERS.

HERBERT CROSSLEY ASHWORTH, Assistant Engineer and Surveyor to the Crompton Urban District Council, Shaw, near Oldham.

ERNEST JOHN HAMLIN, B.Sc., A.M.I.Mech.E.,

A.M.I.E.E., Town Engineer, Stellenbosch, Cape Colony, S.A.

ARTHUR HANSON, M.R.San.I., General Manager to J. Schofield & Son (Wyke), Limited, Wyke, Bradford.

FREDERICK GEORGE KEENE, Chief Inspector of Mechanical Engineering and Building Trades Departments, Western Electric Company, Woolwich.

OTWAY PRIOR KENNEDY, Assistant Engineer to Chief Engineer for Docks, N.E.R. Company, King George Dock, Hull.

THE PRESIDENT then read his address as follows :—

GENTLEMEN,—In commencing my second presidential address to you, I desire in the first place to express my gratitude for the generous support I have received from the Council and Members during the past year. It has been a year of anxiety—because we did not know from day to day how we might be affected by the progress of the war, but I am happy to say that so far our membership and funds have suffered no material loss. Many of our members are serving with the forces, but we have had a record number of meetings, and they have all been well attended. The massive volume of TRANSACTIONS, issued in May last,* is full of most valuable information, and even country and foreign members must admit that they get a good return for their subscriptions, although they are unable to join in our discussions.

The labours of the Council of the Concrete Institute in the matter of the Reinforced Concrete Regulations, extending over a period of six years, have at last borne full fruition. In the draft of the Regulations, which was submitted to the Council for final approval before they became law, the whole of the Council's recommendations were found to have been adopted, with very slight modifications here and there. Very prolonged attention and a considerable amount of thought and anxiety have been devoted to the subject, and the result is one of which the Institute may justly feel proud. Reinforced concrete construction carried out in accordance with these Regulations will be absolutely safe, and so far as can be judged at present, its effective life will only be measurable by centuries.

* Vol. V. Pts. 3-6.

The completion of these Regulations has been, for us, the great event of the year, and our heartiest thanks are due to the London County Council. No previous Building Regulations have been compiled with so much care or with so honest an endeavour to meet the reasonable views of all parties. The exceptional position which the Institute holds in regard to the subject secured a certain amount of preferential consideration for its views, and our good friend Mr. E. Fiander Etchells was a sympathetic medium.

We have no great work in hand now, but we cannot afford to stand still. If we wish to progress we must look around and see where improvement can be made.

The arrangement of our present committees does not seem to me quite the best that could be desired. For example, the Science Committee should in my opinion be more comprehensive. It should undertake the whole business relating to the principles of, and constructional practice in, reinforced concrete, with power to appoint sub-committees, such as Cement Tests Sub-Committee, Research Sub-Committee, Failures Sub-Committee, besides others from time to time dealing with specific subjects such as Notation, Specifications, Quantities, Pile-Driving, Foundations, etc.

We have a large and representative Council and several Committees of experts, but there is too great a tendency to let the work devolve upon a few willing members. I find that in the last complete yearly record of attendances, while there were five who made an average of over 60 per cent., half the members attended less than 10 per cent. of the meetings.

Many interesting papers have been read and discussed at our evening meetings, and I need not recapitulate them. We are glad to welcome papers in all branches of theory and practice in structural engineering, particularly those relating to practice, and when the author has the courage to state his opinion freely the discussion is always vigorous. There must of necessity be differences of opinion, and it is only by free discussion that we can arrive at the truth. Our motto should be "Try all things, hold fast that which is good."

In view of the present predominance of steel as

a material of construction, and the exceedingly satisfactory way in which it behaves in general, it is somewhat curious to look back and note the distrust with which it was regarded by engineers of the last generation. When Sir Henry Bessemer suggested the use of steel for rails, a prominent engineer remarked that those who wished might use steel, but that personally he was not desirous of being indicted for manslaughter. The good qualities of the material were, however, gradually recognized, but at the same time there were periodically expressions of alarm at certain mysterious failures, of which at the present time we hear little or nothing. When I found that steel was economically displacing iron, and wished to use steel plating in some screw tugs I designed for use on the Thames some years ago, my clients grumbled at what they considered unnecessary extravagance, and remarked to a friend that I "would build them of *platinum* if they would let me." I was, however, able to show a considerable reduction of cost by using steel instead of iron, although I found that for the purpose required, the thickness could not be reduced in full proportion to its increased strength.

M. Osmond has pointed out that the quality of structural steel is really a function of three variables, viz., its chemical composition, its structure, and its freedom from initial stresses. All of these variables may differ materially, even in bars rolled from a single ingot. Experiment shews that while the ingot is solidifying, the carbon, sulphur, and phosphorus tend to concentrate in the centre of the upper third of the ingot. Indeed, a content of phosphorus, on the average but 0·06 for the whole ingot, may, through segregation, be five times this figure in parts. The structure of a steel depends mainly on its treatment from the ingot up to the finished bar. Much light has been thrown on this matter by the microscopic examination of the polished and etched surfaces of the metal, whilst the knowledge of the properties of solutions worked out in other departments of science have permitted of some explanation being given of the peculiarities of structure found. Mere heat treatment is alone sufficient to alter considerably the structure of a bar. These facts are now pretty well known, and

the engineer is seldom much troubled about the quality of his materials, since most steel works are in a position to supply an article of uniform and excellent quality.

We have not yet settled the relative position of the architect and engineer in steel-framed and reinforced concrete buildings. Mr. Cocking says: "Another aspect which should not be lost sight of is the status of the engineer in the future. The 1909 Amendment is essentially an engineers' Act, and the reinforced concrete regulations will be more so; therefore, it seems within the possibilities of the near future that, provided the engineer takes advantage of his opportunities, he might assume the more important position, and in such case the architect would confine his attention solely to the architectural treatment—unless, of course, the architect more fully appreciates the rapid change in modern building conditions and construction." Mr. Brown says that an architect is necessarily a structural engineer, and I agree that if an architect is to have the responsibility for the stability of his building, he ought to that extent to be a structural engineer, but what is the fact in the majority of cases? If Sir Christopher Wren had been a structu al engineer we should have seen various modifications in the structure of St. Paul's Cathedral.

Mr. Brown is very emphatic when he says: "It is the architect, and the architect alone, who should determine the positions of all main girders, stanchions, and supports." If the architect is wise, he will welcome any suggestions the designer of the steelwork may be willing to make. It is very nice to have a clear area without stanchions, but this is a luxury that has to be paid for heavily; a stanchion in the middle practically reduces the stress on the girder to one-fourth.

An editorial in *Concrete and Constructional Engineering* says: "Generally speaking, it is the architect who will settle the construction to be employed, leaving out for the moment those schemes which are executed by the contractor direct for the client, and it is necessary that this fact should be realized at the outset, because it has an important bearing on the subject. Now, if architects as a body could be convinced that an actual saving is effected by the use of reinforced concrete, there is no doubt that

structural steelwork would at once take second place in all important work, but there are difficulties to be overcome which must not be overlooked. In the first instance, architects are familiar with steelwork, and can generally calculate the required strength of ordinary members with the aid of available tables, and they have a certain feeling of confidence in dealing with a material which they understand. With reinforced concrete the circumstances are different, there being extremely few architects who thoroughly understand the theoretical side of this material, and there are no tables or 'rule-of-thumb' methods for making even approximate calculations, and as they cannot be expected to take up the subject in the same manner as the engineer, and become proficient in the design, they are apt to avoid the material as much as possible. Secondly, the architect is not limited by any patent system if he decides to use steelwork. But with reinforced concrete he is limited to a certain extent by systems, by specialists who wish to take off their own quantities, by licensed contractors, and by other questions which are not helpful. If he selects a particular firm of specialists he eliminates competition, which he must have for the satisfaction of his client, and the only alternative is to employ a consulting engineer, who will prepare a scheme upon which to obtain estimates. This method is by far the most satisfactory, but it is not always possible to persuade the client to pay the necessary fees to the engineer, and the architect cannot afford to pay them himself. We quite agree with Mr. Watson's remarks as to the great waste which occurs when several firms are asked to prepare calculations and schemes for the same building, when only one set of drawings will be used. If it were only possible to accomplish, it would pay all the specialists to combine, in the case of competition schemes, and jointly employ one independent engineer to prepare a set of drawings upon which each firm could tender. The result would be lower prices and a proportionate increase in the use of reinforced concrete, which would eventually be beneficial to all the firms."

I have previously expressed the opinion that there should be more collaboration between the architect

and the engineer, and our joint meetings will go far to remove any difficulties that may arise in practice. It is said in some quarters that there is too great a tendency for architects to look upon concrete specialists and constructional designers as merely better-class craftsmen, but as craftsmen to be entirely subordinate to the architect. I can only say that architects are among my best friends, and I cannot complain that they have shown any lack of consideration in any views I have put before them.

Although this has been one of the most eventful years in the history of the world, there is not very much to chronicle in the domain of reinforced concrete construction. Of the larger buildings in this material that have been completed during the year, there is the very large and effective warehouse built for Messrs. Ralli Brothers, at Stanley Street, Manchester. Another large warehouse is that at Laisterdyke, Bradford. A two-million gallon reservoir at Runcorn and many smaller works and buildings have also been built on the same system during the year.

Owing to the moist atmosphere to be resisted, reinforced concrete is exceptionally advantageous for swimming baths ; good examples will be found at Croydon and Hammersmith. For water towers it has also many advantages, although perhaps the tank portion could be constructed more cheaply in steel alone. A water tower at York, said to be the largest in Great Britain, was constructed to hold 300,000 gallons. Many other interesting works have been carried out, but if you are subscribers to that useful monthly magazine, *Concrete and Constructional Engineering*, you will be kept informed of what goes on.

A most unusual piece of engineering construction is nearing completion at Baltimore. The work consists essentially of converting an old brick-lined water tunnel, in which the flow was by gravity, into a pressure conduit operating under a head of 35 lbs. per square inch. The novelty lies in the use of huge separately moulded reinforced concrete pipes, the largest having a diameter of 9 ft., to form the new tunnel lining. As the existing tunnel is only 12 ft. in diameter, it is clear that the work had to be carried on in cramped quarters, and, in addition to the difficulties inherent in placing

$10\frac{1}{2}$-ton sections of pipe with so little working room, there is the obligation of shutting down operations every few days and using the conduit to deliver water to the city. The valves are then closed and the tunnel drained in order to allow the work of lining to proceed. One of the features of the construction is a specially designed car, which carries the ponderous pipes into the tunnel, and by means of adjusting equipment sets them accurately to line and grade. The concrete pipes are of particular interest, for they are claimed to rank among the largest precast units ever built. It is also unusual to use reinforced concrete to resist such a great internal pressure as is found in the Baltimore tunnel conduit.

The *Engineer* reports that in connection with the enlargement of the Victoria terminus of the London, Brighton, and South Coast Railway, a covering of fine concrete to all the overbridges in the station has been adopted. The ironwork is thus protected from corrosion from the weather, and from the steam emitted by the locomotives passing under it. At the Cortland Street ferry end of the Pennsylvania and New York City Railway, the whole of the steelwork has been covered with a thin coating of sand and cement, sprayed on it by means of a "cement gun" after carefully cleaning the surfaces of the steelwork. A test piece successfully resisted the most severe trials. It was kept in moist salt air for three days, in salt water for three days, in a temperature of 5° below zero for forty-eight hours, and then allowed to thaw in a warm room; then it was placed on a boiler subjected to a heat of 110° for three days; then it was dropped 2 ft. on a wooden floor, again on a concrete one, and showed no signs of cracking.

The *Engineering Record* of New York, in a recent leading article, states that every war affords great scope for the skill of the engineer, and the present terrific struggle has already presented some questions of startling significance, reference being made to the gigantic tasks of transportation which are being performed. Not less in importance, however, are some other problems which demand immediate attention for efficiency and solution. One of these is to find the material of maximum effective resistance to high explosives. It seems to have been demonstrated effec-

tively that concrete, which has been the main standby in modern fortifications, is almost useless in its customary form. The turrets of permanent concrete forts were rendered useless by a few rounds of shells charged with high explosives. From the photographs of the demolished works it appears that, even when the turrets themselves were not demolished, they were fairly blown off their foundation by the complete shattering of the concrete bases. On the other hand, temporary earthworks, and one or two forts heavily supported with earth, appear to have resisted attack by high explosives measurably well. A shell penetrating a few feet into concrete rends the whole mass, while in earth it merely opens a crater, which is partly filled in again by the falling dirt. It was noticed in the American Civil War that an active shovelling party could in a short time repair the damage of a heavy bombardment where an earthwork was concerned. The questions now to the front are whether any reinforcement of the concrete will be of service, or whether a proper choice of earth construction will so muffle the explosives as to make them comparatively ineffective. A few feet of loose dirt, or even of snow, will stop a bullet which would pierce a half-inch iron plate, and the same principle seems to hold good for heavy projectiles. Lieut.-Col. Roustam Bek, the talented Russian expert, whose interesting war articles in the *Daily Express* may be known to many of you, says: "Engineers do not know any kind of material stronger than concrete and steel for cupolas and other bulwark constructions, but those, as far as we can see, are not sufficiently solid to withstand the terrible fire of the monstrous 16-inch guns. It has always been so, and it is at present, that the technical progress of the artillery has been more rapid than the progress of the engineering art, and the rôle of the fortress will be unsatisfactory unless its fortification be made impervious to the destructive action of siege artillery."

It is possible, however, that it may be found useful for the interior of forts with sufficient earth protection, as it has already proved valuable in roofing trenches and dug-outs where the earth can be placed on top, and also for side walls in fighting trenches when sufficient time is available.

In this country we have found reinforced concrete

useful for huts for training camps, and a large number of them have been constructed with expanded metal. The scare last autumn, of the concrete gun platforms put in secretly by the enemy, and discovered at various strategical points in England, appears to have evaporated, at least so far as the public are concerned. Whether there was anything in it can well be left in the hands of our military advisers, but the number that have been actually discovered on the Continent makes it probable that the scare was well founded. I have also just read the following in the *Daily Express* (Sept. 14th), showing what preparation was made even on the Italian frontier. "On only one section of the Alpine frontier barrier, from the Stelvio Pass (9,050 ft.) on the Swiss frontier to the Adamello glacier, a distance of about sixty miles, the Austro-German Alpine clubs, subsidized by their Governments, built splendid roads up to the summits of the passes and peaks, and built two hotels and several 'huts.' These huts are large enough to contain at least twenty 'tourists,' and their ground floors were constructed of cement. It is very rare to find twenty Alpinists together in a high Alpine hut, and cement floors are a luxury never met with in the Swiss Alps, and therefore could only have been meant for military purposes. Italian Alpinists rarely visit these parts, and nothing was known of the armoured Alpine huts, which continue all along the zigzag frontier along the Dolomites and Carnic Alps, to near Gorizia, where a few men in the huts with Maxims can hold a considerable force at bay."

With further reference to the effect of reinforced concrete in military operations the following is *a propos*. A writer appointed by the Home Office to observe and describe the effect of the Zeppelin air-raid over London on October 13th, in the course of his description, said :—

"The third area contains two damaged business premises, the first of them a large and modern building constructed of reinforced concrete, and with a steel and concrete roof and flooring. Two bombs dropped in this building, one of them actually on the roof, and one on the pavement immediately beneath the doorway.

The bomb on the pavement appears to have exploded sideways ; at any rate, the damage done, which consisted chiefly of broken glass and plaster, occurred mainly in the houses on the other side of the street. The bomb which dropped on the roof of the building itself did little damage."

This information is very satisfactory in view of the very considerable damage done when an explosive bomb dropped on a brick building.

In the reconstruction of the numberless bridges on the Continent destroyed by the war, reinforced concrete will no doubt be largely used. It is a fine opportunity, and I hope some of our members will, later on, be able to give us a good paper on the subject. A reinforced concrete bridge at Soissons was blown up by the Germans in their retreat from that town, but, owing to the great strength and toughness of the material, it defied all attempts at complete destruction, such as was readily effected in the case of the other bridge at Soissons, which was of metal. The former still crosses the river after a fashion, and gangways laid over the ruins give access to both sides.

Another instance of the resistance of reinforced concrete to explosives was given in June last, when an attempt was made to blow up a building in Ontario, where military clothing was being made for the British Army. From information gained by the authorities the charge consisted of twenty-six sticks of dynamite, but the only result was to blow out ten to fifteen feet of a brick retaining wall, damage a concrete slab forming a footway, and shatter the glass of fifty to seventy-five window sashes. The owner expressed the opinion that if the building had been in any other material than reinforced concrete, it would have been a total wreck.

In my previous address I called attention to the fireproof qualities of reinforced concrete construction, and during the past year these qualities have been exemplified on more than one occasion. According to the *Architects' and Builders' Journal*, an intense fire at Messrs. Davis and Sons' tannery at Kingston, Ontario, demonstrated very clearly the eminently fire-resisting character of reinforced concrete construction.

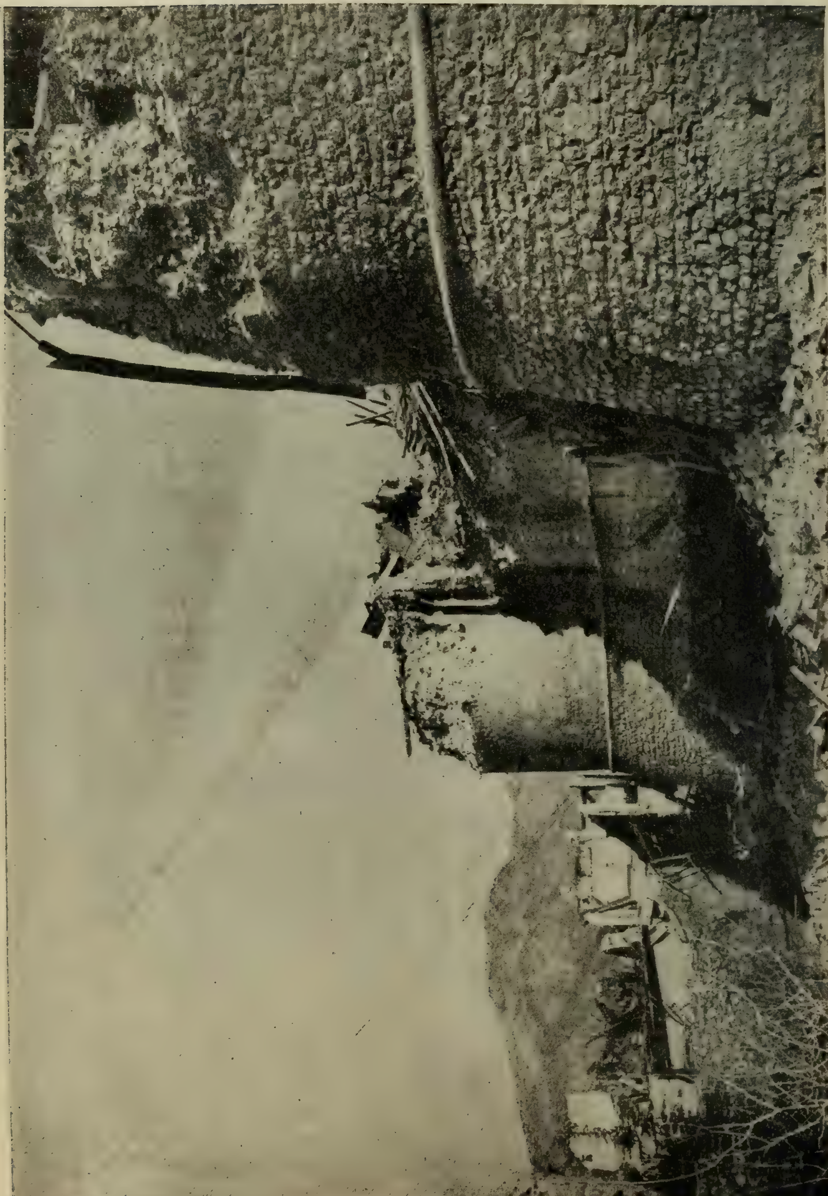
These works consisted of several so-called fireproof buildings adjoining a reinforced concrete building. It is a remarkable fact that whereas all the other buildings were completely destroyed, the reinforced concrete building remained uninjured, although it was close up to the buildings that were burnt down. A four-story brick and steel building was completely destroyed, and of the brick engine-house nothing was left but a mass of ruins. There was a leach-house on the site, with reinforced concrete walls built around the interior structure, which consisted of steelwork within hollow tile floors ; the reinforced concrete walls stood up, but the interior collapsed. A few days after the fire the owners had made good the slight injury to the reinforced concrete building and continued work in it.

A fire in a six-story warehouse of reinforced concrete at Youngstown, Ohio, is believed to have been caused by an explosion on the sixth floor, where two car-loads of matches were stored. For three hours these matches burned furiously, the temperature reached being 1,700°. In spite of this furious fire, a careful examination of the roof construction immediately above revealed practically no injury to this portion of the building. On the fifth floor the injury to the reinforced structure was but trifling, and in other parts of the building the structural injury was inappreciable, although fierce fires of highly inflammable materials had occurred in them. Immediately after the fire a test load of 300 lbs. per square foot was placed over all the areas, and the maximum deflection of a beam was found to be only one-sixteenth of an inch. Although the value of the contents destroyed was £40,000, the injury to the building was made good at an expense of less than £500.

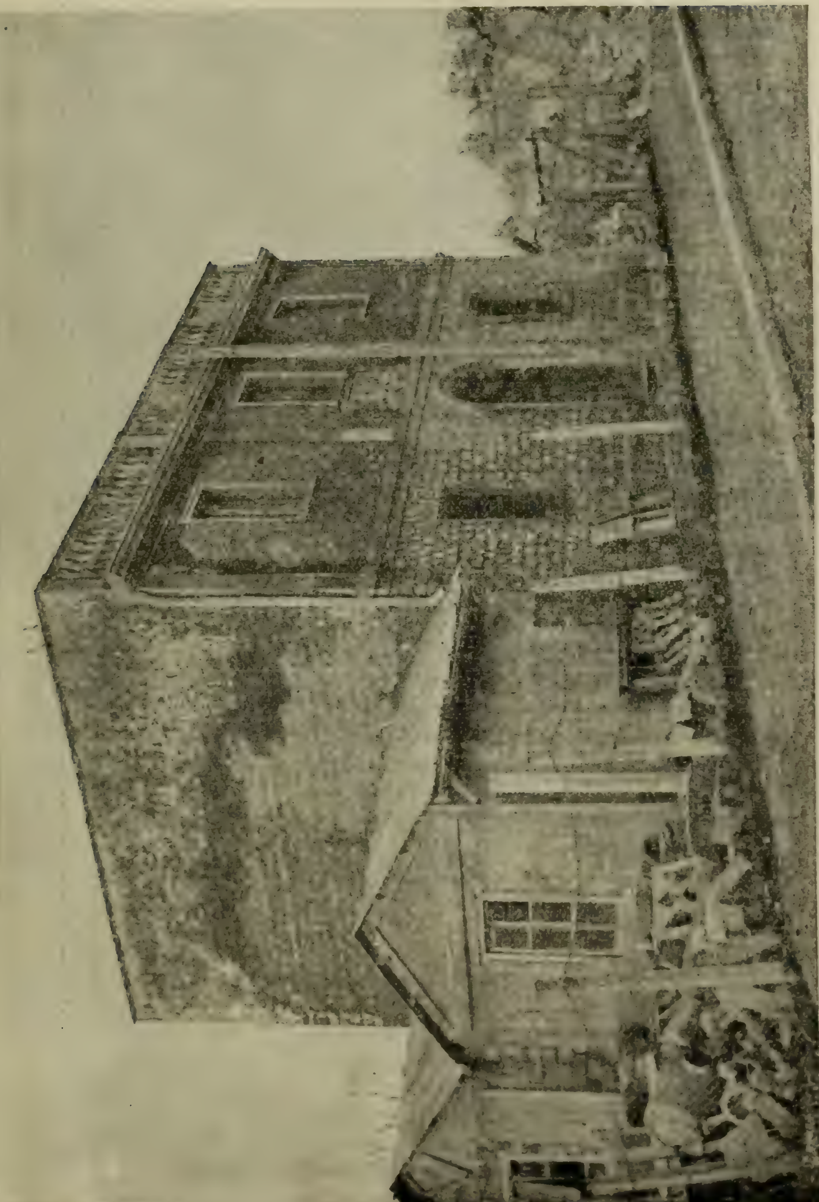
Further testimony as to the stability of reinforced concrete buildings under the action of fire comes from the accounts of the fire which destroyed a large portion of the plant of Mr. Thomas A. Edison, at West Orange, New Jersey, on the 9th of December last. The first account erroneously stated that the concrete buildings, supposedly fireproof, had been destroyed. These accounts were, according to the *Architects' and Builders' Journal*, very inaccurate. The seven con-

crete buildings attacked still stand, while about them were the tangled and hopeless ruins of one frame and six brick buildings. The fire was a very fierce one, and five chief reasons are assigned for the extent of the disaster and the rapid progress of the fire. These are : (1) The highly inflammable character of the contents of the buildings ; (2) the inadequacy of the water supply ; (3) the fact that the window openings were fitted with wooden sashes and plain glass ; (4) lack of fire walls ; (5) lack of automatic sprinklers. With the exception of the upper floors of certain sections the concrete buildings remained in good condition and the salvage was proportionately extensive. Columns spalled here and there, without, however, exposing the reinforcing rods, and occasionally, but not often, the beams also spalled. In general, the more destructive effect of the fire upon the columns than on the beams was noticeable. The effect of the fire on the columns extended generally to a depth of about $1\frac{1}{2}$ in. Except where the fire was very severe, the concrete did not come off and the rods were not exposed. Longitudinal cracks were found on many of the columns, the reinforcing of which consisted only of four vertical rods without horizontal ties or hooping. A considerable proportion of the damage to the columns is attributed to the streams of water which were directed upon them when heated, and it is the belief of the officials that very little damage would have been done had the water not been directed upon them after they had become very hot. Three specialists employed in the investigation have reported that $87\frac{1}{2}$ per cent. of the concrete construction remained in first-class condition and the work of restoration was promptly put in hand. No better recommendation of reinforced concrete could be given than the evidence afforded by this fire.

In my former presidential address I also referred to the stability of reinforced concrete against earthquakes. In the earthquake at Avezzano in Italy on January 13, 1915, out of 12,000 inhabitants only a few hundreds escaped immediate death. Avezzano was situated on the slope of a supposedly extinct volcanic crater, which for centuries had been filled with water, and was known as Lake Fucino. In the middle of the last century



Orsini Castle, Avezzano, Italy: Built in A.D. 1400 and destroyed by earthquake 1915.



The only house left standing by the earthquake in Avezzano: it was built of reinforced concrete.

a rich papal banker, Prince Torlonia, went to enormous expense draining the lake. An area covering forty square miles became a fertile agricultural land, devoted to raising sugar beet, and dotted with scores of prosperous villages and towns, of which Avezzano was the chief. To-day only one building remains intact. This is a house built of reinforced concrete on the road from the station, and owing to the kindness of the Trussed Concrete Steel Co., Ltd., I am enabled to show you a view of the Castle of Orsini at Avezzano destroyed by the earthquake, although the walls were 6 feet thick and had stood all shocks for five hundred years ; together with the reinforced concrete building which remained standing. These views are reproduced by permission of the *Daily Mirror*.

Of course we are all interested in the serious development of the cement industry, and the progress of building in concrete and reinforced concrete, but sometimes there is a humorous side which obtrudes on our view. We have heard of the United States being described as "the land of promise," and many of us are familiar with the "Eden" described in "Martin Chuzzlewit," but it has been left to one of its latter-day inhabitants to create a new Eden within the borders of that great country, which gives a meaning to the name more approximate to that mentioned by Dickens than to its scriptural prototype. It forms one of the oddest homes in the world and is to be found in Lucas, Kansas. The old gentleman who owns this home has aimed to reproduce the original Garden of Eden in cement. The house is of the log-cabin style, and is built of stone logs, some of which are 20 ft. long. The porches, walks, fence, and trees are all made of cement, no wood whatever being used. Near the cabin the owner has built a cement mound for a strawberry bed. The plants grow from holes in the sides. Forty-two tons of cement were used in the construction of this "Garden of Eden." As the visitor enters the front gate he sees Adam and Eve with arched arms over the walk, and Eve handing an apple to Adam. An immense serpent is twined on one of the trees watching the apple transaction. There are seven cement trees from 30 ft. to 38 ft. high, and twelve others from 8 ft. to 15ft. high. On one of the corner trees are

to be seen Cain and his wife, and on the next tree, Abel (dead) and his wife, while a dog is in an angle of the tree. In one tree is the "All-seeing Eye and Hand," while in another Cain and Abel may be seen making their offering (pumpkin and sheep), while near by are two girls. To the right of the two girls are two storks roosting, and back of them is the figure of the devil. The devil measures 8 ft. in height. He is holding his tail with one hand and a fork with another. The place presents a weird appearance at night when lighted up, as the owner has installed electric lamps in the mouths of the various animals.

Owing to the shortage of German and Belgian supplies of Portland cement, a good deal of attention is being given to the manufacture of this material abroad, in places where limestone, shale, or clay and coal can be obtained of suitable quality and in sufficient quantities to make the manufacture likely to be successful and remunerative. The Queensland Cement and Lime Company has now arranged to establish works at Darra, near Brisbane. The plant is designed to produce about 40,000 tons of cement per annum. The complete contract for the supply and erection of the plant, which will be of entirely British manufacture, has been placed with Messrs. Noyes Brothers, Sydney. Grinding mills of the combination ball and tube mill type, and a kiln 140 ft. long and 8 ft. in diameter, will be erected; and these, together with all the subsidiary plant, will be electrically driven, direct-coupled wherever possible. The motors are for 3-phase, 50 cycles, 440 volts, and will be supplied with power from generating plant in the works, which will be direct-coupled to vertical high-speed engines. The general lay-out of the plant has been made with the expectation that the works will soon be duplicated and even further increased.

Why is it (says the *Engineering Record*) that in two concrete buildings apparently constructed under identical conditions, built by contractors of equal intelligence and integrity, from concrete composed of similar aggregates and the same brand of Portland cement, the floors in one will turn out hard, firm, and resistant to abrasion, while in the other ordinary usage will result in dusting? The fact that numerous dustless

concrete floors have been laid seems to indicate that the trouble must lie in the selection, proportioning, mixing, placing, or finishing of the material. The procedure and proportions described below have given excellent results. For first-class work a rich mixture is desirable, say, one in which the aggregate consists of granite, or other hard stone, screenings graded from $\frac{1}{4}$ in. down to the finest, and crushed stone of equal quality passing a $\frac{1}{2}$-in. ring and retained on a screen having $\frac{1}{4}$ in. mesh. All trowelling and finishing of the floor surface should be completed within two and a half hours from the time the materials leave the mixer. This necessitates mixing the material to such consistency that the mortar has to be scraped from the wheelbarrows, and will hardly flatten out when dumped upon the floor wet enough so that it can be "struck off" with little difficulty when spread out with shovels. The floor usually is in a condition to be trowelled for the last time within an hour and a half or two hours after the finishing course has been mixed. When sufficiently hardened to prevent pitting, the floor should be sprinkled with water until 2 in. of sawdust can be thrown on the surface without injury. The sawdust should be thoroughly wetted and kept moist by sprinkling for a period of two weeks.

A mixture of 15 or 20 lbs. iron dust with 100 lbs. Portland cement added to twice that quantity of sand applied as a top coat 1 in. thick to a concrete floor is said to render it hard and durable.

What may be called the standard mixture for concrete in reinforced work is 1 part Portland cement, 2 parts sand, and 4 parts larger aggregate, by volume, commonly known as a 1 : 2 : 4 mixture. There is a very objectionable practice on the part of some people, who ought to know better, to call this a 1 to 6 mixture, with the result that a builder may have some ground for assuming that he complies with the requirements by using 1 part of cement to 6 parts of combined sand and larger aggregate, which would approximately be a 1 : 3 : 6 mixture, but there is no guarantee of the proportion unless the materials are mixed separately, 90 lbs. of cement being taken as equal to 1 cubic foot.

In a curious case that came to my knowledge

gas fumes were given off from some coke breeze concrete floors. The cause seems to have been that the coke breeze was not completely carbonized before being used, and the gas left in it was liberated by the action of the lime pugging filled in between the flooring fillets. The smell was always stronger on humid days ; this was accounted for by the pugging being able to hold a considerable amount of moisture and so increasing the chemical action. It is another illustration showing the wisdom of following the advice of the Concrete Institute, and not using coke breeze concrete at all.

In the sample of concrete before you, you will see pieces of coal and various other indications of a bad mixture. There are several points to be learnt from it, and it will repay a careful examination.

The study of photo-micrographs of concrete shews that, although standard proportions of the materials are used, the resulting mixture will not be perfect unless these proportions are suited to the voids in the aggregate, and it thus becomes essential to ascertain the voids in any proposed aggregate before proceeding farther. The quantity of water required in mixing concrete will depend somewhat upon the materials, but it appears from a series of experiments that the best proportion is 25 to $27\frac{1}{2}$ per cent. of the mass, although this is rather wetter than we usually make it.

In view of the remarks upon failures of concrete structures in my last address, the paper read by Mr. R. S. Greenman before the American Society for Testing Materials, and published in the last December number of *Concrete and Constructional Engineering*, is particularly interesting. He stated that the causes approximated to 90 per cent. due to poor workmanship, 8 per cent. to poor aggregates, and 2 per cent. to poor cement. Fortunately, the larger the work the greater the care that is taken, so that the failures chiefly lie with the smaller and less important constructions. He gave instances of failure due to impure water, sand, and stone, all of an exceptional character, and for that reason more liable to escape attention. He considered the failures due to poor workmanship to be so common as not to need detailed examination

and summarized the chief appearances of improperly mixed concrete. In conclusion, he recommended both laboratory tests and field inspection, so that the conclusions of the investigator might be founded upon a consideration of the failure from every point of view, whether the object of the examination was to place responsibility or to guard against future failures. From the engineer's point of view the latter is the sole point of interest.

In a case of defective workmanship, thirty thousand cubic feet of water shot in a cataract from a tank on the roof of a mill at Bradford on February 12, 1914, when one of the end walls collapsed, and swept scores of bales of wool out of the building. The mill, known as Cashmere Works, was completed six months before. It was built of reinforced concrete, and was 150 ft. long, 70 ft. wide, and about 150 ft. in height, divided into six stories. Across the entire top of the building was a tank capable of holding 3 ft. of water for the fire-sprinkling apparatus. One end of the building burst outwards with a loud crash, the water-tank collapsed, and the water rushed through the building and out into the street, carrying with it many tons of wool and cashmere. The street slopes considerably, and the water poured down in a flood so deep that bales of wool floated along for two hundred yards. No workmen were in the part of the building where the accident happened, but only a little while before a large number had left. Practically the whole of one end of the building was torn out by the weight of wool and the force of the water.

The *Engineering Record* says: "There are too many things falling down for the welfare of engineers and contractors alike. The concrete factory roof that fell in Ilion and the concrete roof that fell in St. Paul recently because the supports were removed too soon, the corner of the brick building in Boston because of rotten foundations, the front of the brick building which fell in Chicago because there was little to keep it in place, the various other structures that have been dropping conspicuously from their place in the landscape, are precursors of a fall in the reputation of engineers and contractors if they do not

look alive. It is their misfortune that their successful works are without interest to the man in the street, while their unsuccessful works attract attention. There is no field of highly trained endeavour where accidents have more serious consequences, and so there should be no field where more general attention is directed to preventing such accidents."

The time has gone by when engineers are justified in regarding their knowledge of how to do things as professional secrets. Looking at the matter strictly from the standpoint of individual selfish advantage, and not that of professional welfare, it is not desirable to have the same mistakes repeated. Engineers are not usually retained by other engineers, but by men without technical knowledge. These men, learning that certain kinds of work or certain classes of plants have not been successful, are naturally prejudiced against them, and look with suspicion on the advice of any engineer on such subjects. The best way to have the engineering profession strengthened is to prevent inferior or bad engineering work, and the best way to accomplish this is to tell freely what proves successful and what proves unsuccessful, so that everybody may profit, so far as in him lies, by the knowledge of all. No engineer will lose standing among his colleagues or be ranked any less competent by business men for frankly putting on record a statement of his non-successes. On the contrary, he will receive the approbation of all thinking men, for he will warn others away from a repetition of his unsuccessful work, which is a most desirable thing.

In Mr. R. S. Greenman's paper we read among the opening remarks, "Concrete is said to be its own best inspector, and it is a well-known fact that defects in concrete will sooner or later make their presence known." Now, I do not think much of an inspector who only makes defects known when it is too late to remedy them, but the important point is that defects will shew themselves sooner or later. He goes on to say, "For every fault there must be a reason." That sentence ought to be taken as "the text" by our Investigation Committee, and no case should be allowed to rest until the "reason" or primary cause is found.

We must not be discouraged if we are unable to

learn as much as we should wish. One of the greatest men who ever lived, Sir Isaac Newton, when congratulated by his friends upon the extent and greatness of his discoveries, replied : " I know not what the world will think of my labours, but to myself it seems that I have been but as a child playing on the shore, now finding some brighter pebble, now some more variegated shell, while the vast ocean of truth lay unexplored before me."

SIR HENRY TANNER, C.B., I.S.O., F.R.I.B.A., F.S.I. (Past Pres.C.I.), proposed a vote of thanks to the President for his interesting address, and in the course of his remarks, said, that most bodies follow the lead of the Concrete Institute with respect to matters cognate to the interest of the Institute, and this is rather a feather in its cap. As regards the relations between the Architect and the Specialist Engineer in steel-framed and Reinforced-Concrete buildings, he added, that the architect's point of view needs consideration ; the latter gets only 5 per cent., and cannot be expected to supply for this percentage much information in the way of detail to the Specialist Engineer ; this is, of course, a defect in the present method of procedure, nor must those cases be forgotten, where the Architect pays the Engineer's fees.

MR. E. FIANDER ETCHELLS, Assoc.M.Inst.C.E., F.Phys.Soc., A.M.I.Mech.E. (Member of Council, C.I.), in seconding the vote of thanks, adverted to the President's remarks as to the work of the various Committees, and said that the art to be cultivated was that of so filing and co-ordinating the information obtained, as to be able to lay hands promptly upon any item of such information. He would ask Professor Adams to tell them what system of classification he would recommend, or what is the system he employs, as to enable him to retain his youthfulness so successfully. Would Mr. Hills, too, tell the Meeting what are his experiences of the results of bomb damage.

MR. OSBORN C. HILLS, F.R.I.B.A., District Surveyor for the Strand, replied, that he is collating the information upon the subject.

The President replied to the vote of thanks, which was heartily accorded by those present.

SIXTY-FIFTH ORDINARY GENERAL MEETING

WEDNESDAY, JANUARY 19, 1916

THE SIXTY-FIFTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held at Denison House, 296, Vauxhall Bridge Road, Westminster, London, S.W., on Wednesday, January 19, 1916, at 5.30 p.m.

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., M.S.A., F.S.I., F.R.San.I., etc.), in the Chair.

The following elections took place :—

MEMBERS.

FRANK LURCOTT PAWLEY, Assoc.M.Inst.C.E., Civil Engineer, Assisting on the Hull and Barnsley Railway, at present Captain R.G.A., Sunk Island Battery, Hull.

JOHN CHARLES TELFORD, Assoc.M.Inst.C.E., M.J.I.E., Civil Engineer ; and Assistant Manager, Messrs. Dorman, Long & Co., Ltd., London.

WILLIAM EDWARD WOOLLEY, M.S.A., Architect's Chief Assistant, Teacher of Building Construction, etc., in Evening School, London.

ASSOCIATE-MEMBERS.

EDWARD SMITH COLDWELL, A.R.I.B.A., P.A.S.I., Architect and Surveyor, London.

CHARLES WILLIAM GLOVER, M.J.I.E., Assoc.Fellow Perm. Way Inst. ; Draughtsman, Gen. Civil Engineer, etc., Chief Engineer's Office, Port of London Authority, London.

WILLIAM GREEN, Inspector of Works, Northern Lighthouse Commissioners, Cape Wrath Lighthouse, Durness, Scotland.

The Meeting was devoted to a Discussion of the appended Report of the Joint Committee on Loads on Highway Bridges. The Report was discussed and considered by the Meeting, and it was decided that it should be published as a separate Report after dis-

cussion and after the delegates of various Societies had considered it.

(N.B.—Owing to the disturbance to normal conditions, caused by the Great War, and as one result this Volume not being published till after the conclusion of peace, the Editor is able to announce here that the above Report is now available to the public in its final form, and can be obtained from the offices of the Concrete Institute.)

LOADS ON HIGHWAY BRIDGES.

REPORT OF JOINT COMMITTEE.

At the request of the Council of the Concrete Institute the Institution of Municipal and County Engineers and the Institution of Municipal Engineers agreed to a joint conference with them upon "The Loads to be provided for upon Highway Bridges." The following delegates were appointed :—

Representing the Concrete Institute.

Professor Henry Adams, M.Inst.C.E., etc.
Mr. William Dunn, F.R.I.B.A.
Mr. Charles F. Marsh, M.Inst.C.E.
Mr. C. S. Meik, M.Inst.C.E.

Representing the Institution of Municipal and County Engineers.

Mr. John A. Brodie, M.Eng., Wh.Sc., M.Inst.C.E.
Mr. A. E. Collins, M.Inst.C.E.
Mr. J. W. Cockrill, M.Inst.C.E.

Representing the Institution of Municipal Engineers.

Mr. A. E. Prescott, M.R.San.I.
Mr. Henry C. Adams, M.Inst.C.E., M.I.Mech.E.
Mr. H. C. H. Shenton, F.S.E., M.R.San.I.

Secretary of Committee: Mr. H. Kempton Dyson,
Secretary C.I.

and met on December 1, 1911, to arrange a plan of procedure.

Professor Henry Adams was elected Chairman. Mr. W. C. Copperthwaite, M.Inst.C.E., the London County Council

Bridge Engineer, and Mr. H. Percy Boulnois, M.Inst.C.E., late of the Local Government Board, were co-opted to assist the Committee.

Inquiry forms were issued to the various County, City and Borough Surveyors, Railway Engineers, Contractors, and others, soliciting information upon the heaviest vehicles in use in their neighbourhoods and the loads they carried.

The responses to the circular were very numerous, for which the Committee were grateful, but were rather slow in coming in, and some time was necessarily occupied in digesting them.

On October 16, 1912, the Chairman submitted a summary of the information obtained, together with a draft report and a preliminary classification of bridges according to the loads for which they should be designed. This draft included a discussion of the question of width of wheel tyres and diameter of wheel for various loads, but as the Committee decided that this matter was outside the present inquiry it has been inserted as an Appendix. The Classification of Bridges was fully discussed by the Joint Committee, and was then condensed so as to give only three classes, and various other modifications have been made from time to time, resulting in the accompanying notes, diagrams, and table of loads, which the Committee recommend as giving suitable provision for all classes of traffic.

In designing bridges with cross girders the latter must be sufficient to carry the maximum axle load travelling across the centre of their span at the given wheel gauge, together with the dead load according to the spacing of the cross girders, and a distributed external load of 1 cwt. per foot super over the area not occupied by the rolling load.

In designing arched bridges the curve of thrust should be drawn when the heaviest axle has reached the centre of the nearer haunch, and again when it has reached the centre of the further haunch. These two positions will probably give the greatest variations in the stress to be provided for. If a third condition be taken it should be when the heaviest axle has reached the centre of the bridge.

As regards the distribution of point loads through the thickness of the road material, it is considered that the spread each way may be taken as equal to the depth, so that the pressure due to a wheel load of 5 tons, on a width of tyre of 3 in., with a depth of road material of 12 in., would be

spread over an area of 2 ft. 3 in. × 2 ft., or be reduced to $\frac{5}{2\cdot25\times2}=1\frac{1}{9}$ tons per sq. ft. on the under surface.

As to the effect of speed in increasing the virtual loads, it was stated at a conference of Local Authorities of Surrey and Middlesex, held at Richmond in September 1912, that "Engineers' calculations showed that the weight on each back wheel when the (motor) omnibus was loaded and standing was $2\frac{1}{2}$ tons. At a speed of 10 miles an hour the force equalled $4\frac{1}{2}$ tons, and at 15 miles it equalled 7 tons." Upon plotting the above values and extending to 20 miles per hour, as in Fig. 1, the improbability of their accuracy will be apparent. At 25 miles per hour the $2\frac{1}{2}$ tons dead load would have become about 20 tons.

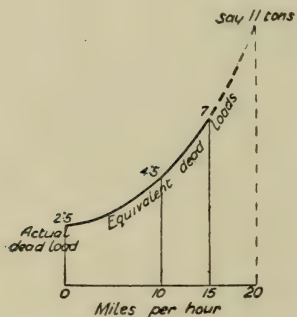


FIG. 1.

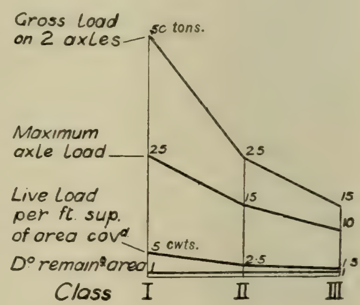


FIG. 2.

In a paper published in the *Journal of the Western Society of Engineers* for June 1913, Professor Dufour has shown by data derived from experiments that a critical speed exists at which the vibration and stresses induced by a rolling load reach a maximum. The critical speed is different for various kinds of traffic and also depends upon the class of bridge. So far the information available is insufficient to formulate any practical rules by which the designer may determine, even roughly approximately, the critical speed for a bridge, but it is suggested that experiments should be made to determine the critical speed for any particular bridge under test loads by which it would be possible to regulate the maximum permissible speed for various kinds of traffic on the bridge.

TABLE I.

SUGGESTED STANDARD LOADING TO BE PROVIDED FOR IN DESIGNING HIGHWAY BRIDGES.

(See Fig. 2 and Fig. 3.)

Class.	Description.	Wheel Base.	Gross Area covered by each Vehicle.	Maximum Axle Load.	Gross Load on Two Axles.	Load per ft. sup. on gross Area covered.	Load on remaining Area including Footways.
I	Bridges on main thoroughfares in great industrial centres and within a radius of 25 miles thereof	All approximately 10 × 6 ft., and 10 ft. between wheel bases of different vehicles	All approximately 20 × 10 ft.	One vehicle of			1 cwt. per foot super to cover other traffic at same time (Only one of the maximum loads given on next page should be on the bridge at one time.)
				25 tons	50 tons	5 cwt.	
				together with a tractor of			
				12 tons	22 tons	2·5 cwt.	
	Will take ordinary boiler trollies and heavy guns			Total space occupied 40 × 10 ft.			
II	Bridges on secondary roads in great industrial centres and on main roads outside a radius of 25 miles			Two vehicles of			
				15 tons	25 tons	2·5 cwt.	
				together with a tractor of			
				12 tons	22 tons	2·5 cwt.	
	Will take 20-ton rollers, large traction engines, large ploughing engines, L.C.C. tramcars, etc.			Total space occupied 60 × 10 ft.			
III	Bridges on provincial roads other than main roads		18 × 9 ft.	Tractor of			
				10 tons	15 tons	1·5 cwt.	
				together with 3 trailers each			
				7 tons	12 tons	1·5 cwt.	
	Will take 15-ton rollers, ordinary traction engines, ploughing engines, brick trucks, siege guns			Total space occupied 72 × 9 ft.			

Note.—For those places in the North of England and Midlands requiring special bridges (such as Leeds, Sheffield, West Bromwich, Salford, etc.) a maximum axle load of 35 tons or a gross load of 70 tons on two axles with a wheel base of 10 × 6 ft. should be provided for.

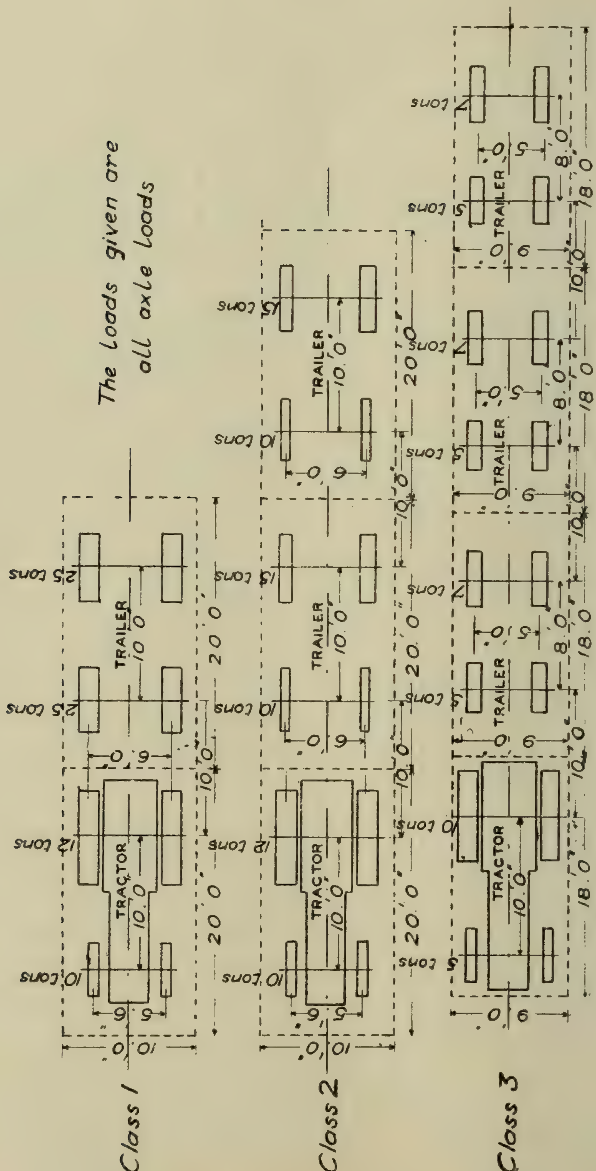


FIG. 3.—Typical Maximum Loads for each Class of Bridge.

The records of such tests would eventually enable formulæ to be deduced for calculating the critical speed.

The condition of the road surface would have more influence than the speed. The vibration of the load, and consequent increase of stress, will probably vary with the irregularity of the surface, and this will bear a definite proportion to the effort required for traction, an estimate of which is given below.

TRACTIVE EFFORT REQUIRED ON DIFFERENT ROAD SURFACES ON THE LEVEL.

Asphalt	40 to 50 lbs. per ton of load.
Paved road (good order)	60 „ 80 „ „
„ (fair order)	80 „ 100 „ „
Macadam road (dry and good order) ...	100 „ 120 „ „
„ (wet and good order) ...	130 „ 150 „ „
„ (fair order)	170 „ 200 „ „
„ (muddy)	220 „ 250 „ „
„ (loose stones)	300 „ 350 „ „

That a rolling load does produce a virtual increase in its intensity by the vibration due to the irregularity of the surfaces in contact is undoubtedly a fact, but it is very difficult to determine what allowance should be made for it. No general rule could be more than an approximation, as the effect will vary with the ratio of the weight of moving load to that of the structure supporting it, *e.g.* on a small railway bridge 50 per cent. must be added to the weight of a locomotive in order to find the equivalent dead load to produce similar stresses, while on such a structure as the Forth Bridge even the dead weight of the locomotive might be ignored, as it forms so small a part of the total load.

It is probable that if bridges are designed in accordance with the recommendations of the Committee, with a factor of safety of 4 for spans over 50 ft. and 5 for smaller spans, the loads may all be taken as dead whatever the speed may be at which they are travelling. The dead load will consist of the whole weight of the superstructure, including the road material, curbs, footways, parapets or handrails, tramway tracks, gas and water mains, sewers, and any other permanent addition that may be required.

As regards the sufficiency of the Committee's Standards for Military purposes, the Director of Fortifications and Works writes from the War Office under date March 16, 1916, "I am directed to inform you that the classification proposals appear suitable to War Department requirements."

NOTES ACCOMPANYING TABLE OF STANDARD LOADING FOR BRIDGES, PARTICULARLY WITH REGARD TO HEAVY LOADS.

There are two standards adopted in Leeds, viz.—

Class 1, for all bridges on arterial main roads, is based upon a maximum axle load of 35 tons and a total load of 70 tons on a 10×6 ft. wheel base. Out of the 170 bridges within the city boundaries, 42 are included in this class.

Class 2, for all bridges on non-arterial main roads, is for a maximum axle load of 25 tons and a total load of 50 tons on a 10×6 ft. wheel base. There are 29 bridges included in this class, which corresponds to the first class in the preceding table.

In designing any road bridges the gradient of the approaches and the locality generally are considered in fixing the load to be taken into account.

The Manchester Ship Canal bridges are calculated for a maximum axle load of 30 tons, or 50 tons gross, on a 10×5 ft. wheel base.

At West Bromwich the heaviest boiler trolley has carried a gross load of 55½ tons, and a double tandem boiler trolley 56¼ tons.

At Hull the last bridge, a reinforced concrete arch bridge, was designed for a 50-ton load on a 10×6 ft. wheel base, travelling at 3 miles per hour, the footpath being loaded simultaneously with 150 lbs. per foot super.

At Wakefield the largest boiler trolley is designed for a gross load of 100 tons on a wheel base 8×7 ft. The bridges are designed for a 15-ton traction engine and 40-ton boiler trolley on four wheels.

The Middlesex County Engineer is of opinion that bridges should be designed for carrying two steam rollers back to back, 15 tons each, or two fully loaded tramcars, 12 tons each, or a boiler trolley with heaviest load (he mentions a case where a load of 130 tons was carried on sixteen wheels), and as many people as can crowd on the remaining space.

At Ashton-under-Lyne the Borough Surveyor says two boiler trollies, each of 32 tons gross load, may pass each other on a bridge in that district.

At Salford a gross load of 50 tons is provided for on ordinary bridges, but near the Ship Canal Docks a load of 60 tons on four wheels is provided for.

The standard load adopted by the Manchester Corporation for bridges is a 30-ton trolley on 8 ft. 6 in.×6 ft. wheel base

of four wheels, or 60 tons on eight wheels, with 17 ft. centres of trolleys. An additional live load of 80 lbs. per foot super over the remainder of bridge is provided for.

The Derbyshire County Council provide for a 22-ton traction engine and 48-ton boiler trolley.

At Cardiff the bridges, where wide enough, are designed for two 15-ton rollers passing each other, or for a 16-ton traction engine and 32-ton boiler trolley.

Holdsworth & Sons' boiler trolley has carried a gross load of 40 tons, the rear axle load being about 25 tons.

The boiler trolleys may be drawn by a traction engine weighing 15 tons, with a rear axle load of 10 tons on a wheel base of 10×6 ft. and overall length of 15 ft., but in some cases the weight of a traction engine reaches 22 tons.

American traction engines of the largest size vary from 26,000 lbs. total weight for 25 h.p. to 40,860 lbs. for 30 h.p., the rear axle with the driving wheels carrying from half to four-fifths of the total load. The recommended allowance in American practice for heavy county bridges is 200 lbs. per sq. ft. of floor surface, or a 20-ton traction engine with axles 11 ft. centre to centre and with wheels 7 ft. apart on the rear axle ; four-fifths of the weight of the engine is carried on the rear axle.

PRACTICAL APPLICATION OF THE STANDARD LOADING TO THE DESIGNING OF GIRDER BRIDGES.

In the following tables exact accuracy has been sacrificed to convenience, but it is believed that the figures are approximately correct and sufficiently near for the purpose of ascertaining the stresses due to a live load of the standard laid down for each class of bridge in Table I.

Main Girders.

The bending moments, Table II and Figs. 4 and 5, are for the whole live load, irrespective of the number of girders, and the shear stresses, Table III and Fig. 6, are based upon the same assumptions. In narrow bridges, if there are two main girders the moments will be equally divided over each, supposing the bridge floor to be continuous. In the case of wide bridges, where the load may be sometimes nearer one main girder and sometimes nearer the other, the division o

TABLE II.

MAIN GIRDERS.

Bending Moments for Rolling Loads (in Tons-Feet).

Central Ordinate for Simple Parabola.

Maximum Bending Moment, with whole load on one girder. No allowance for impact.

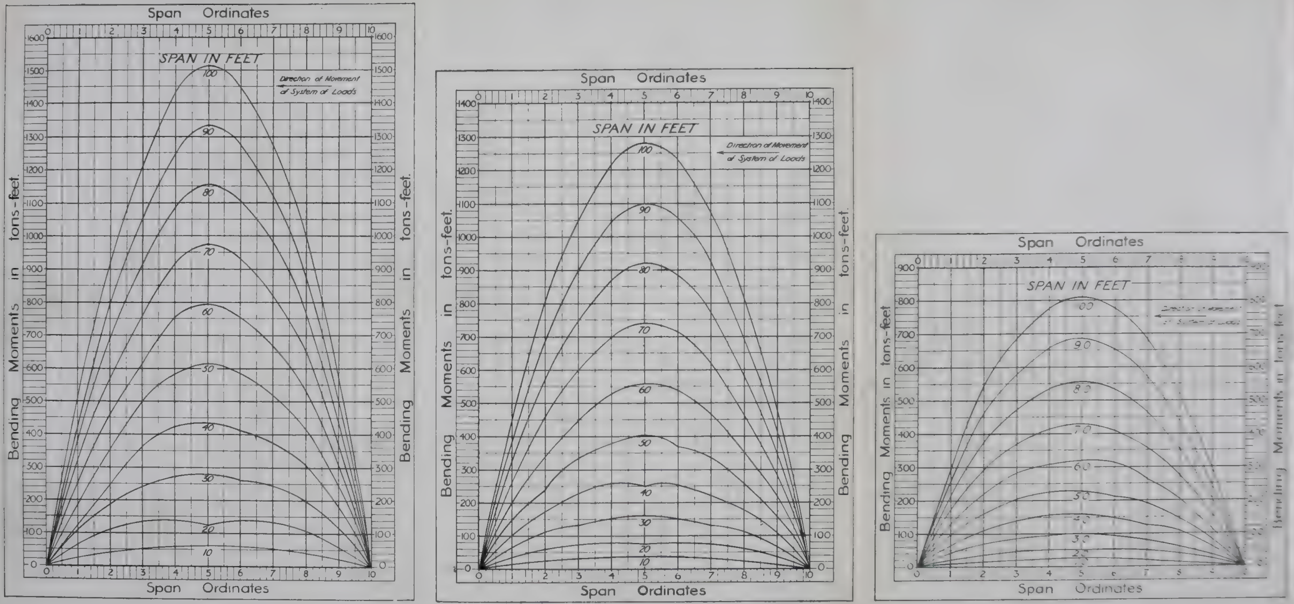
Span in Feet.	10.	20.	30.	40.	50.	60.	70.	80.	90.	100.
Class 1	62·5	170*	285	435	615	795	973	1,155	1,335	1,513
Class 2	40·0	85†	162·5	270	405	560	740	920	1,100	1,280
Class 3	26·0	55	100	163	235	328	430	558	685	810

* Maximum at 0·37 span, viz. 140. Actual at centre 123. Central ordinate to enclosing parabola 170.
† Maximum at 0·4 span, viz. 80. At centre 75. Central ordinate to enclosing parabola 85.

TABLE III.

Shear Stresses for Rolling Loads (in Tons).

Span in Feet.	10.	20.	30.	40.	50.	60.	70.	80.	90.	100.
Class 1—										
Sc	12·5	12·5	16·7	18·75	21·2	22·8	24·8	26·3	27·2	28·1
Sab	25·0	37·5	45·7	52·25	56·2	58·8	60·7	62·1	63·2	64·1
Class 2—										
Sc	7·5	7·5	9·17	10·0	12·0	13·3	15·0	16·25	17·8	19·2
Sab	15·0	20·0	26·0	32·5	38·4	44·0	48·0	51·0	53·3	55·2
Class 3—										
Sc	5·0	5·0	5·2	5·35	6·2	7·1	7·8	8·65	9·5	10·4
Sab	10·0	13·2	16·5	19·8	23·0	26·4	29·6	33·0	35·8	38·0



Class I.

Class II.

Class III.

FIG. 4.—Bending Moment Diagrams.

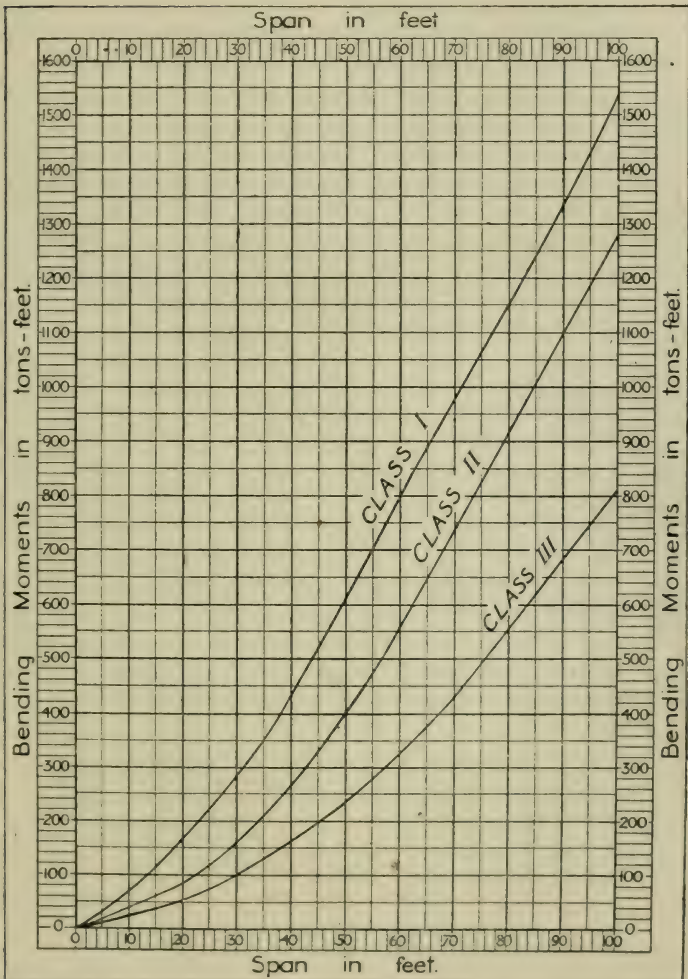


FIG. 5.—Graph of Maximum Bending Moments on Main Girders.

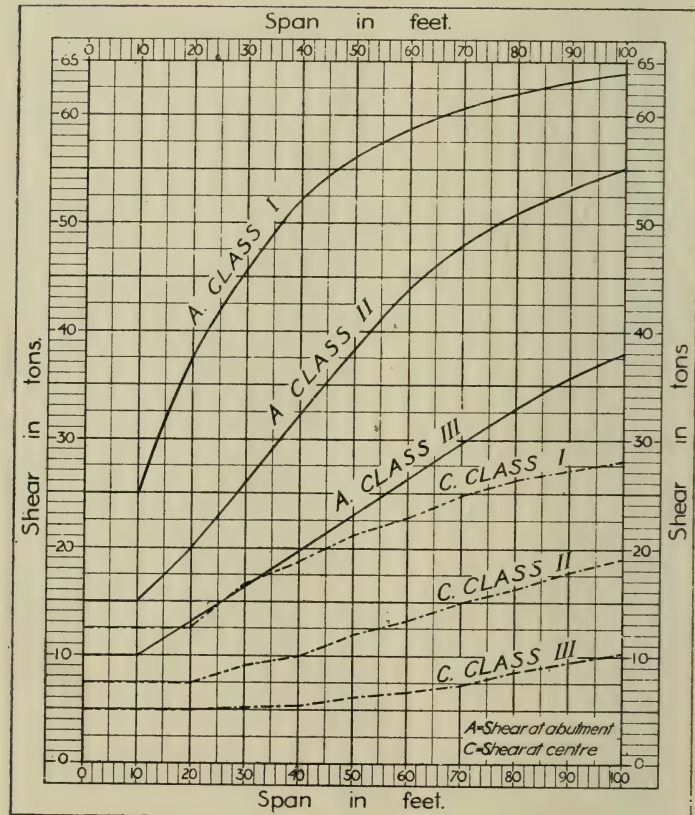


FIG. 6.—Graph of Shearing Forces on Main Girders.

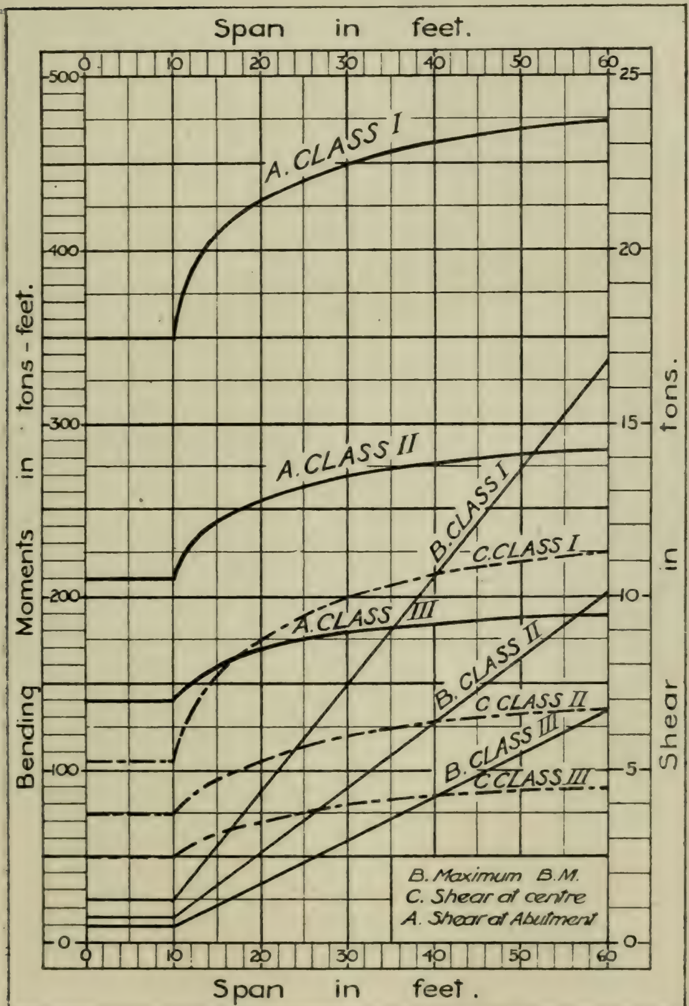


FIG. 7.—Graph of Bending Moments and Shearing Forces on Cross Girders.

TABLE IV.

Cross Girders.

Bridges.		
Class 1	12·5 tons ↓— 6 ft.—↓ 12·5 tons	Change of position equivalent to a rolling load.
Class 2	7·5 tons ↓— 6 ft.—↓ 7·5 tons	
Class 3	5 tons ↓— 6 ft.—↓ 5 tons	

Span in Feet.	10.	20	30.	40.	50.	60.
Class 1—						
Max. B.M. ton-ft.	25·0	87·5	150·0	212·5	275·0	337·5
Shear centre	6·25	8·75	10·0	10·625	11·0	11·25
Shear ends	17·5	21·25	22·5	23·125	23·5	23·75
Class 2—						
B.M. centre	15·0	52·5	90·0	127·5	165·0	202·5
Shear centre	3·75	5·25	6·0	6·375	6·6	6·75
Shear ends	10·5	12·75	13·5	13·875	14·1	14·25
Class 3—						
B.M. centre	10·0	35·0	60·0	85·0	110·0	135·0
Shear centre	2·5	3·5	4·0	4·25	4·4	4·5
Shear ends	7·0	8·5	9·0	9·25	9·4	9·5

$M_c = \frac{W(l-d)}{4}$, $S_c = \frac{W}{2} - \left(\frac{W}{2} \times \frac{d}{l}\right)$, $S_{ab} = W - \left(\frac{W}{2} \times \frac{d}{l}\right)$, d = centre to centre of axles,

the bending moments and shear stresses should be proportioned to the possible requirements. If there are cross girders the bending moments are modified as given in Table V.

Cross Girders.

The bending moment and shear stresses, Table IV and Fig. 7, allow for the live load crossing at any part of the span.

Dead Loads.

The dead loads on all girders must be allowed for in the usual way.

Contingencies.

If it be desired to allow for the possibility of a traction engine or motor vehicle mounting the curb and running on the footway, the side girders must be designed to take a larger proportion of the load than they would ordinarily bear.

TABLE V.

Effect of Cross Girders on Bending Moments of Main Girders.

The effect of cross girders in dividing up the load on a main girder is to give the same maximum bending moment as a uniformly distributed load when the number of bays is even, and a reduced bending moment when the number is odd, as in the following table.

No. of Bays.	Max. B.M.	Percentage.
1	$W\,l/8$	100
2	$W\,l/8$	100
3	$W\,l/9$	88·8
4	$W\,l/8$	100
5	$W\,l/8{\cdot}3$	96
6	$W\,l/8$	100
7	$W\,l/8{\cdot}16$	97·96
8	$W\,l/8$	100
9	$W\,l/8{\cdot}1$	98·76
10	$W\,l/8$	100

APPLICATION OF TABLES.

Note.—Bending moments and shear stresses from structural load to be added in each case.

Example A.

Class I.—100 ft. span, 80 ft. wide, three main girders, cross girders, 8 ft. centres, footway 10 ft. wide.

MAIN GIRDERS (100 ft. span).

Central Girder (Table II).

Maximum bending moment from vehicular live load, by table, 1,513 ton-ft. (as it may have to carry nearly the whole). The width contributing its load to the central girder will be 40 ft., of which 10 ft. is occupied by the rolling load, leaving 30 ft. in the calculation. As the whole of the rolling load has been taken on the main girder it will be safe to neglect the live load on the strip before and behind the rolling load. From live load of 1 cwt. per foot super over remaining area (excluding before and behind the above load)—

$$\frac{W\,l}{8} = \frac{30 \times 100 \times \frac{1}{20} \times 100}{8} = 1{,}875 \text{ ton-ft.}$$

Together 3,388 ton-ft.

Side Girders (Table II).

Maximum bending moment from vehicular live load, say $\frac{25}{40}$ of tabular value $= 1{,}513 \times \frac{25}{40} = 946$ ton-ft. From live load of 1 cwt. per foot super over remaining area—

$$\frac{\left(\frac{35}{40} \text{ of } 10 + \frac{10}{40} \text{ of } 20\right) 100 \times \frac{1}{20} \times 100}{8} = 859 \text{ ton-ft.}$$

Together 1,805 ton-ft.

SHEAR STRESS in same proportion as bending moments.

Central Girder (Table III).

Shear at ends :

From rolling load (per table) ... ... 64·1 tons.

Half distributed load $= 30 \times 100 \times \frac{1}{20} \times \frac{1}{2} = 75$ tons.

Total 139·1 tons.

Shear at centre : From rolling load (per table) 28·1 tons.
From distributed load ... zero.

Side Girders (Table III).

Shear at ends:

$$\frac{25}{40} \times 64{\cdot}1 + \left(\frac{35}{40} \times 10 + \frac{10}{40} \times 20\right) 100 \times \frac{1}{20} \times \tfrac{1}{2} = 74{\cdot}5 \text{ tons.}$$

Shear at centre $\frac{25}{40} \times 28{\cdot}1$ tons $= 17{\cdot}56$ tons.

Cross Girders (40 ft. span. Table IV).

Maximum bending moment from vehicular live load, by table, 212·5 ton-ft. From live load over remaining area—

$$\tfrac{1}{4} \times 8 \times \frac{1}{20}(18^2 + 12^2) = 46{\cdot}8 \text{ ton-ft.}$$

Together 259·3 ton-ft.

Shear Stress.

Shear at ends:

$$23{\cdot}125 + 8 \times \frac{1}{20} \times \frac{32^2}{2 \times 40} = 23{\cdot}125 + 5{\cdot}12 = 28{\cdot}25 \text{ tons.}$$

Shear at centre $10{\cdot}625 - \frac{8}{20} \times \frac{18^2 - 12^2}{2 \times 40} = 9{\cdot}725$ tons.

Example B.

Class II.—90 ft. span, 60 ft. wide, two main girders, cross girders, 10 ft. centres, footway 10 ft. wide.

Main Girders.

Maximum bending moment from vehicular live load—

$$\frac{45}{60} \times 1100 = 825 \text{ ton-ft.}$$

From live load over remaining area—

$$\frac{\left(\frac{55}{60} \text{ of } 10 + \frac{20}{60} \text{ of } 40\right) 90 \times \frac{1}{20} \times 90}{8} = 1{,}139 \text{ ton-ft.}$$

Together $= 1{,}964$ ton-ft.

Shear at ends:

$$\frac{45}{60} \times 53{\cdot}3 + \left(\frac{55}{60} \times 10 + \frac{20}{60} \times 40\right) 90 \times \frac{1}{20} \times \tfrac{1}{2} = 89{\cdot}35 \text{ tons.}$$

Shear at centre $\frac{45}{60} \times 17{\cdot}8$ tons $= 13{\cdot}35$ tons.

Cross Girders.

Maximum bending moment from vehicles 202·5 ton-ft. From live load over remaining area—

$$\tfrac{1}{4}\times\frac{1}{20}\times 10(28^2+22^2)=158{\cdot}5 \text{ tons.}$$

Together = 361 ton-ft.

Shear Stress.

Shear at ends :

$$14{\cdot}25+10\times\frac{1}{20}\times\frac{52^2}{2\times 60}=14{\cdot}25+11{\cdot}27=25{\cdot}52 \text{ tons.}$$

Shear at centre $6{\cdot}75-\frac{10}{20}\times\frac{28^2-22^2}{2\times 60}=5{\cdot}5$ tons.

Example C.

Class III.—30 ft. span, 40 ft. wide. As the span is less than the width, there will probably be a series of main girders and deck troughing. Say four main girders, any two adjacent ones carrying the load. The greatest stress will be on the central pair.

Main Girders.

Maximum bending moment from vehicles $\frac{100}{2}=50$ ton-ft.

Shear stress at ends $\frac{16{\cdot}5}{2}=8{\cdot}25$ tons.

Shear stress at centre $\frac{5{\cdot}2}{2}=2{\cdot}6$ tons.

Deck Troughing (13 ft. 4 in. span).

Maximum bending moment 18·9 ton-ft., which may be spread over two bays.

Shear stress at ends 8·2 tons.

Shear stress at centre 2·7 tons.

APPENDIX I.

By HENRY ADAMS, M.Inst.C.E.

(*Submitted to Joint Committee, October* 16, 1912.)

The question of loads on highway bridges may be approached from different points of view, but the primary question would seem to be the effect of a load upon the road surface. In the *Surveyor* for April 21, 1911, a synopsis is given of the very erratic regulations in force in the various counties for different wheel loads and tyre widths, made in pursuance of The Highways and Locomotives Amendment Act, 1878. Upon a comparison of these it would appear that on the average a suitable standard for general use would be 5 cwt. per wheel per inch width of tyre, making no distinction between summer and winter, or between wheels of various diameters. This would, however, be a very unscientific rule, as the damage to a road surface decreases under the same load per inch as the width of tyre increases, and also as the diameter of the wheel increases, but any rules for universal application must above all things be simple and easily applied. After a long and careful consideration of the matter I suggest that the maximum allowable loads should be as given in Table IA, p. 53, which is plotted in Fig. 1A, to enable its regularity to be accurately judged. There should also be a minimum diameter of wheel provided for the loads as in Table IIA, p. 53, which is plotted in Fig. 2A.

These curves practically agree with the formula $\log w = 0{\cdot}96 + \frac{4}{5}\log A - \frac{1}{2}\log d$, where w = width of tyre in inches, A = axle load in tons, d = diameter of wheel in inches. Where tyres are required over 12 in. in width twin wheels should be employed, giving half the width on each.

After long-continued or heavy rainfall, or upon new roads, the maximum axle load given, although well within the mark for ordinary conditions, would probably be found to cause some damage, and some reduction might be desirable if there were any means of enforcing it. Say permission of local authorities for loads exceeding 8 tons per axle except for road rollers, permission not to be withheld, but loads may be reduced to two-thirds normal allowance, in consequence of temporary conditions due to weather. Or simply these loads to be reduced by one-fourth from November to April inclusive.

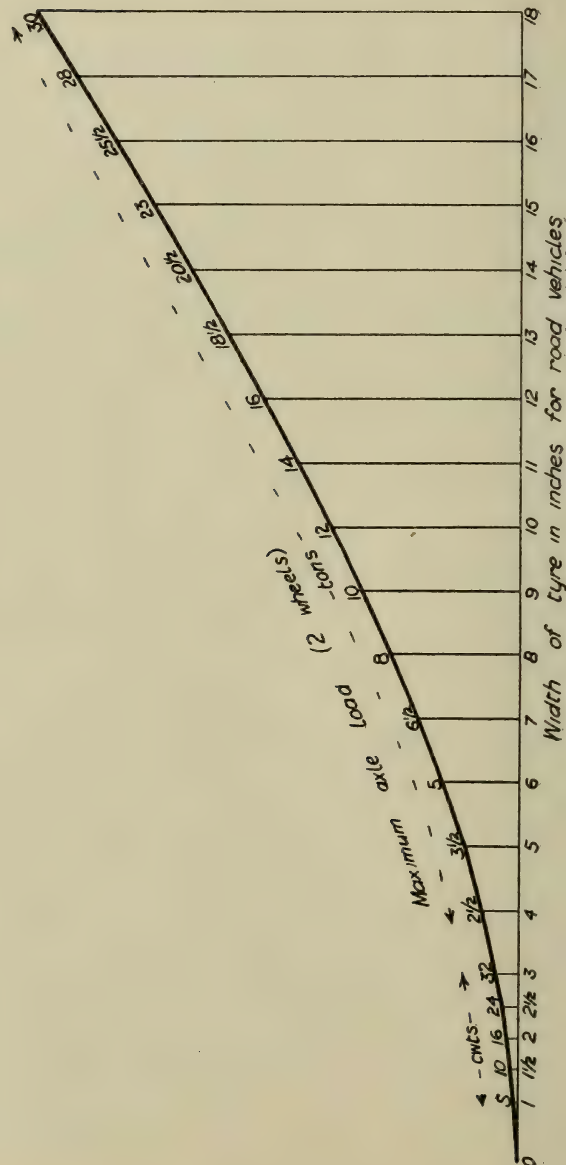

Maximum axle Load (2 wheels) tons
cwts
Width of tyre in inches for road vehicles

FIG. 1A.

A skid-pan or shoe with side flanges (Fig. 3A) should be used when it is desired to lock a wheel while descending a steep gradient, at least $1\frac{1}{2}$ in. wider outside than the tyre and rounded over on the lower edges. The length of its tread should be at least three times the width of tyre with an additional front portion of, say, half width of tyre curved

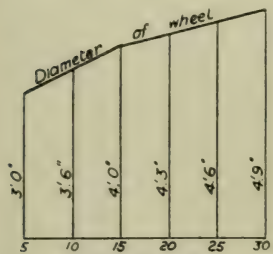


FIG. 2A.

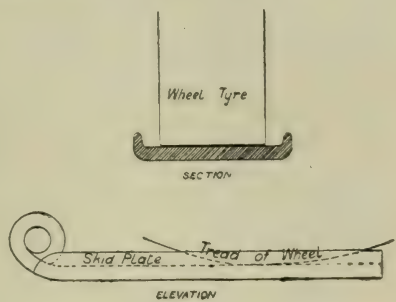


FIG. 3A.

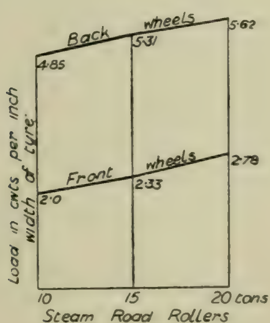
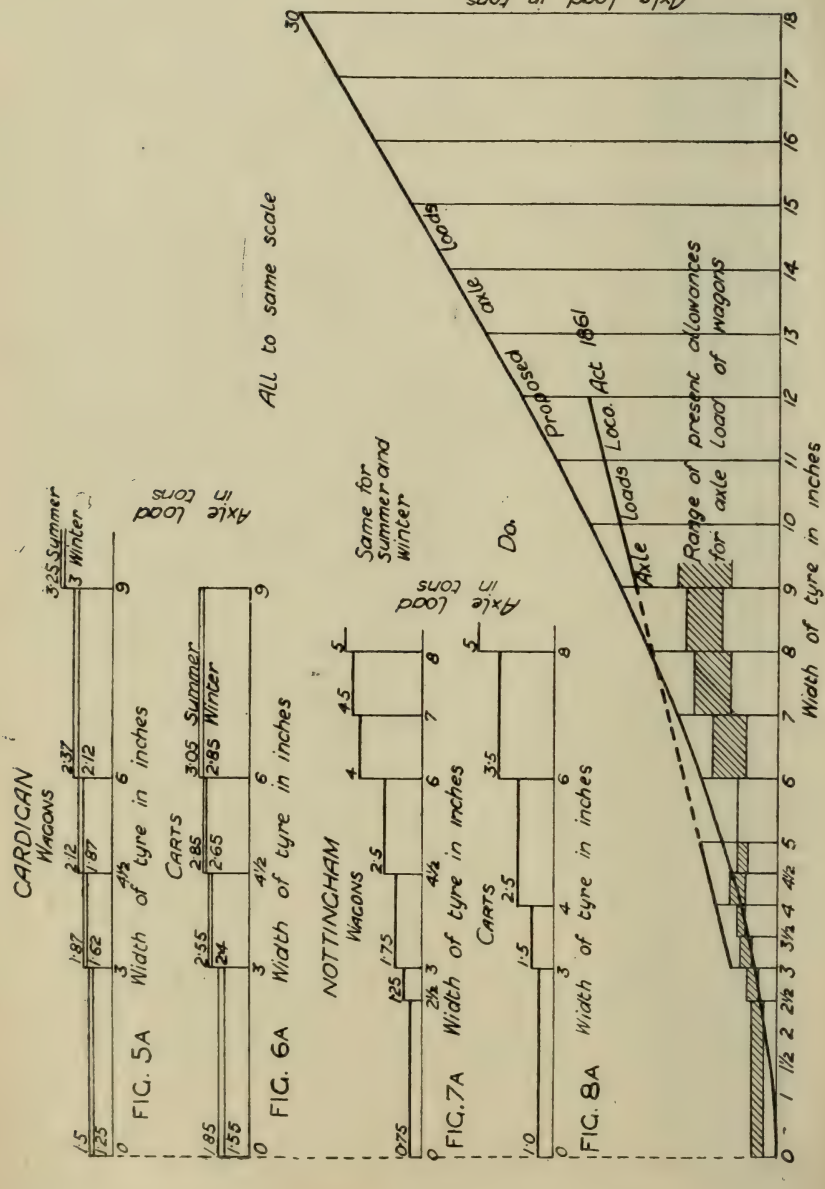


FIG. 4A.

upwards to a height of one-quarter the width of tyre. It should be secured by a strong chain of such length that only one-third of the length of shoe can pass the centre-line of the wheel. Roads are frequently very much cut up by the shoe chain stretching and allowing the fore part of the shoe to dig into the road surface.

Road rollers do not appear to have their loads, wheel diameters, and tyre widths based upon any clear rules, but



Comparative Axle Loads Allowed.

the proportions are more or less regularly graduated in the various sizes. The steam road rollers by Messrs. Aveling & Porter, of Rochester, are typical. Table IIIA, p. 53, gives some particulars of them, which are plotted in Fig. 4A. The principle seems to be that the front wheels have the lighter axle load in order to force the macadam down to interlock without fracture, and the back wheels have the heavier axle load to complete the consolidation of the mass. This back axle load causes about the same pressure upon the road surface as that produced by the passage of heavy traction engines and ordinary loaded boiler trolleys.

It is desirable at this point to see what the official regulations provide for.

A few illustrations based on existing Local By-laws are given in Figs. 5A to 8A, and the whole range of allowances quoted in the *Surveyor* referred to above is plotted in Fig. 9A to compare with the proposed allowances.

Locomotives Act, 1861.

By this Act every locomotive on a highway not drawing any carriage and not exceeding 3 tons in weight shall have

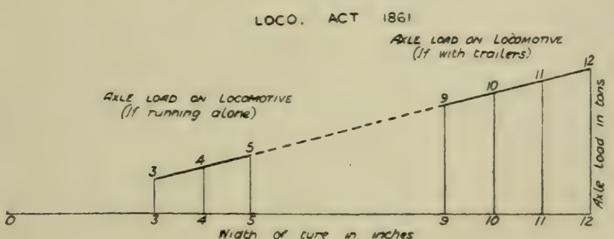


FIG. 10A.

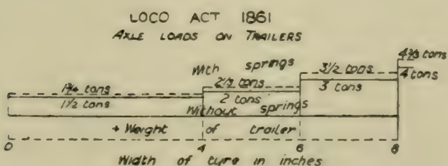


FIG. 11A.

the tyres of the wheels thereof not less than 3 in. in width, and for every ton or fractional part thereof additional the

tyres shall be increased 1 in. in width ; and every locomotive drawing any wagon or carriage shall have the tyres not less than 9 in. in width, but no locomotive shall exceed 7 ft. in width or 12 tons in weight except by permission of the local authority. The wagon or other carriage drawn by a locomotive and having cylindrical wheels shall not carry over and above its own weight any greater weight than
each pair of wheels unless the tyres are 4 in. or more in breadth, nor more than 2 tons unless they are 6 in. or more in breadth, nor more than 3 tons unless they are 8 in. or more in breadth, nor in any case a greater weight than 4 tons on each pair of wheels, unless fitted with springs, when one-sixth more weight is allowed, but this regulation weight shall not extend to the carriage of any article in one block.

These allowances are plotted in Figs. 10A and 11A.

Locomotives on Highways Act, 1896.

This Act applies to mechanically propelled vehicles or "light locomotives" of 3 tons or over, unladen weight, or not more than 4 tons with trailer, both unladen. One trailer only is permitted, and the speed is not to exceed 14 miles an hour.

Locomotives Act, 1898.

This Act allows permission to be given by the County and Borough Councils for loads in excess of those permitted by Section 4, Locomotives Act, 1861. The regulations of weight are not to extend to any wagon carrying a single article weighing more than 16 tons, but the tyres of such wagon shall not be less than 8 in. wide. Not more than three loaded wagons may be drawn at a time without consent. The use of any bridge may be prohibited or restricted if considered unsuited. Only one locomotive shall be upon a bridge at one time.

Motor Car Act, 1903.

By this Act the allowable speed is raised to 20 miles per hour, but on the application of the local authority to the Local Government Board the maximum speed may be limited to 10 miles per hour on certain highways with a view to the safety of the public, and the use of any motor cars may be prohibited on any highway or part thereof that does not exceed 16 ft. in width.

Heavy Motor Car Order, 1904.

By this Order of the Local Government Board, which came into operation March 1, 1905, a heavy motor car is one whose weight unladen exceeds 2 tons, and a trailer is a vehicle drawn by it. A heavy motor car may be used on a highway if it weighs unladen not more than 5 tons, and if with the trailer unladen the total does not exceed 6½ tons, but if it has been registered before the above date it may be used if the weight unladen is between 5 and 7 tons. For military purposes 5 tons and 6½ tons shall be read as 6 tons and 8 tons. The axle weight of a loaded heavy motor car

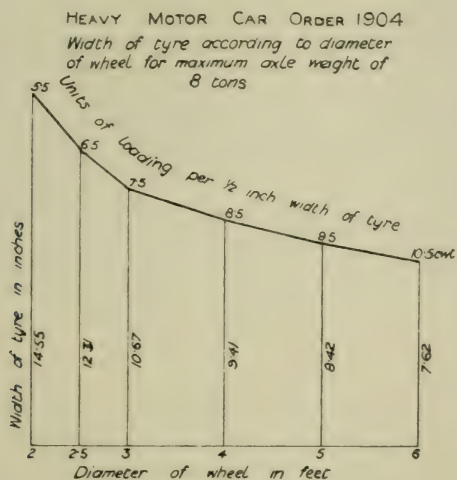


FIG. 12A.

shall not exceed 8 tons, and the sum of the axle weights of a loaded heavy motor car shall not exceed 12 tons.

The width of the tyre of each wheel of a heavy motor car shall not be less than 5 in., or not less than ½ in. for every unit of registered axle weight. The units are 7½ cwt. if the wheel is 3 ft. diameter ; if the wheel exceeds 3 ft. diameter an addition of 1 cwt. for every 12 in. beyond 3 ft., or in like proportions for odd inches ; if the wheel is less than 3 ft. diameter a reduction of 1 cwt. for every 6 in. less than 3 ft., or in like proportion for odd inches. For military purposes the unit of 5 cwt. shall be substituted for 7½ cwt. when the

tyres have diagonal crossbars. The minimum diameter allowed is 2 ft. The same rules apply to trailers exceeding 1 ton in weight unladen, with the substitution of 3 in. for 5 in. as a minimum width of tyre, and 4 tons as the maximum axle weight. But these rules do not apply to wheels with pneumatic tyres or those of soft or elastic material.

The speed at which a heavy motor car is driven on any highway shall not exceed 8 miles an hour, but if the weight unladen exceeds 3 tons, or the axle weight laden exceeds 6 tons, or it draws a trailer, the speed shall not exceed 5 miles an hour.

If a heavy motor car has all the wheels fitted with pneumatic tyres or with tyres made of soft or elastic material, the speed shall not exceed 12 miles an hour when the axle weight does not exceed 6 tons, or 8 miles an hour when the axle weight exceeds 6 tons.

The extreme width of a heavy motor car weighing unladen 3 tons or more, and any trailer drawn by it, may not exceed 7 ft. 6 in. Sufficient springs shall be provided between each axle and the frame of a heavy motor car and its trailer.

A heavy motor car with axle weight exceeding 6 tons shall not be driven upon a bridge forming part of a highway when another heavy motor car or locomotive to which the Locomotives Act, 1898, applies is on the bridge.

The width of the tyre required under the Heavy Motor Car Order, 1904, for the maximum axle load of 8 tons, according to the diameter of the wheel, varies as shown in Fig. 12A. In this case a 3 ft. diameter wheel with an axle load of 8 tons must have a tyre 10½ in. wide, while the proposed rules would for this load make the wheel, say, 3 ft. 4 in. diameter and the tyres 8 in. wide. The transport of loads at the present day requires a much higher allowance than 8 tons for maximum load, and when the Local Government Board regulations are revised provision should be made for a maximum axle load of 30 tons.

The diagrams, Figs. 13A to 26A, represent typical vehicles for different purposes, and Table IVA shows the chief particulars of those vehicles, the most striking feature being the very great irregularity of the intensity of the load per inch of tyre, showing that this by itself cannot possibly provide a basis of classification.

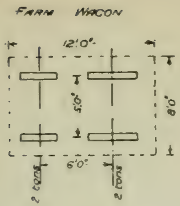


FIG. 13A.

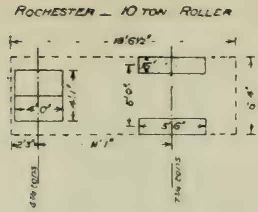


FIG. 14A.

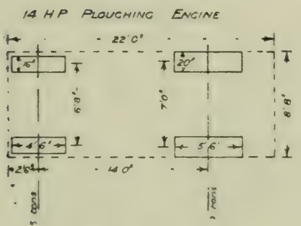


FIG. 15A.

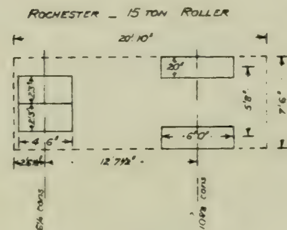


FIG. 16A.

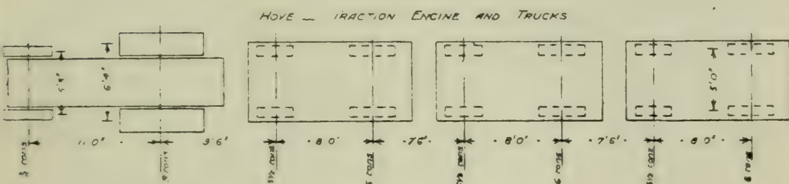


FIG. 17A.

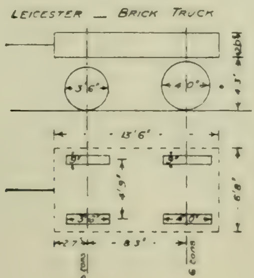


FIG. 18A.

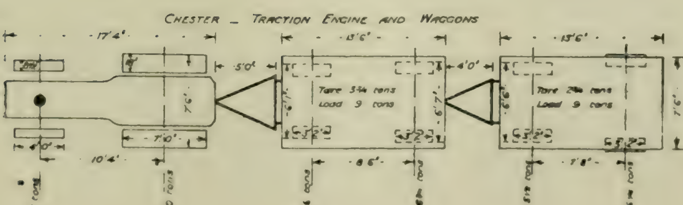


FIG. 19A.

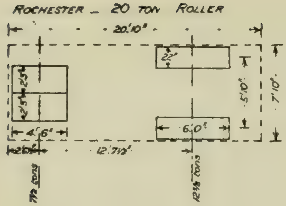


FIG. 20A.

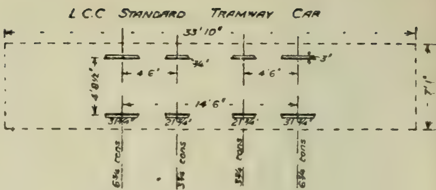


FIG. 21A.

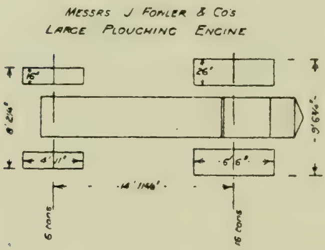


FIG. 22A.

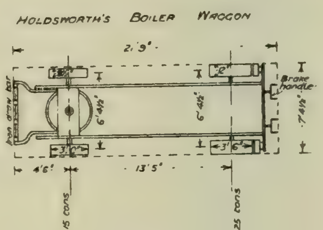


FIG. 23A.

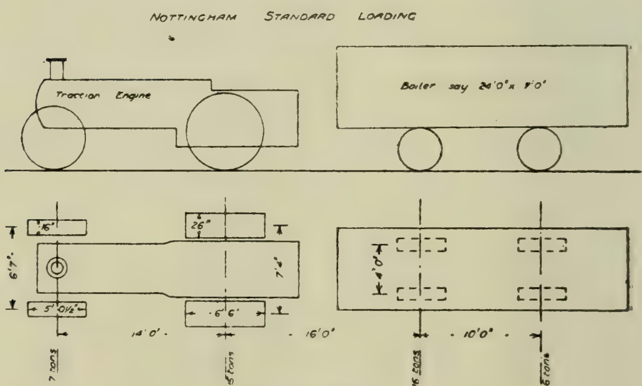


FIG. 24A.

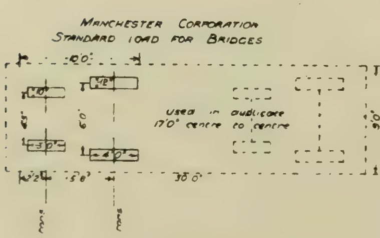


FIG. 25A.

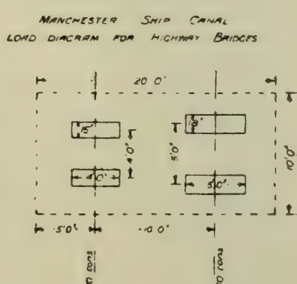


FIG. 26A.

Heavy Motor Car (Amendment) Order, 1907.

The minimum loads to which bridges may be restricted under the Heavy Motor Car (Amendment) Order, 1907, are for a motor car alone a single axle weight of 3 tons or a combined axle weight of 5 tons, or a motor car and trailer together a combined axle weight of 5 tons; and should the combined axle weight exceed 6 tons the motor car shall not be driven upon a highway bridge when another heavy motor car or locomotive under the Locomotives Act, 1898, is on the bridge, but these regulations do not apply to a bridge subject to any special Act or Order, or upon which a temporary prohibition is laid by notice affixed.

To prevent the person liable for the repair of the bridge from improperly alleging that a bridge is insufficient to carry above stated load, it is provided that parties may proceed to arbitration, or if within one month after the request of the owner of the heavy motor car the person liable for the repair of the bridge neglects or refuses to proceed with the arbitration, the prescribed notice on the bridge shall be removed.

A notice permitting a heavier load or loads on the above may be affixed in the first instance or may be substituted for a previous notice.

The following is a typical notice affixed by a railway company to its over-bridges :—

Motor Car Acts, 1896 *and* 1903.

This bridge is insufficient to carry a heavy motor car the registered axle weight of any axle of which exceeds 4 tons, or the registered axle weights of the several axles of which exceed in the aggregate 8 tons; or a heavy motor car drawing a trailer if the registered axle weight of the several axles of the trailer exceed in the aggregate 8 tons.

Name of Railway Company,
Place, Date.

This restriction of the load is only partially modified by the power given to the owner of a heavy car to proceed to arbitration on the ground that the particular bridge is capable of safely carrying a heavier load, but the railway companies are not precluded from substituting heavier loads in the notice if they consider the bridge capable of carrying them. Recent

legislation, however, carries the matter a little further. The Court of Appeal in the case of *Attorney-General* v. *Sharpness New Docks, etc., Company*, decided that a canal company whose special Act bound them to keep the bridges over their canal in "sufficient repair" must maintain them according to an "up-to-date" standard, sufficient to carry the traffic of the day. The same principle was acted upon in the decision of Mr. Justice Warrington in the more recent case of *Attorney-General* v. *Great Northern Railway Company*, in which he held the Company liable to maintain a highway bridge over their railway in such a condition of safety that it would be sufficient for the passage of the traffic which might be expected upon it. The Company had in this case affixed the notice under the Local Government Board Regulations prohibiting heavy motor cars, but the judge held that the true effect of the Regulations was merely to enable the Company to exclude heavy traffic while the bridge remained insufficient, and not to qualify their obligation to render it sufficient if and when traffic might be expected on the highway. In his words "as regards the highway itself, the obligation cast upon the body or person responsible for its maintenance is to maintain it in such a condition that it shall be sufficient for such traffic as may reasonably be expected to pass over it, and is an obligation the measure of which varies with changing circumstances and the habits of those using the road. It may be expressed as an obligation to keep the road up to date." This opinion was held to be a great advance upon many previous decisions, but it was upset upon appeal, and it remains the dictum of the law that a railway company is only called upon to maintain a bridge in a condition *for the traffic at the time at which it was constructed.*

TABLE Ia.

Width of Tyre in Inches.	Maximum Axle Load (2 wheels).	Width of Tyre in Inches.	Maximum Axle Load (2 wheels).
1	5 cwt.	9	10 tons
1½	10 ,,	10	12 ,,
2	16 ,,	11	14 ,,
2½	24 ,,	12	16 ,,
3	32 ,,	13	18½ ,,
4	2½ tons	14	20½ ,,
5	3½ ,,	15	23 ,,
6	5 ,,	16	25½ ,,
7	6½ ,,	17	28 ,,
8	8 ,,	18	30 ,,

For secondary roads add one-third to the width of tyre given above.

TABLE IIa.

Axle Load.	Minimum Wheel Diameter.
	Ft. in.
Up to 5 tons	3 0
,, 10 ,,	3 6
,, 15 ,,	4 0
,, 20 ,,	4 3
,, 25 ,,	4 6
,, 30 ,,	4 9

TABLE IIIa.

Description of Road Roller.	Axle Load in Tons.	Width of each Tyre.	Diameter.	Loads in Cwt. per Inch Width.
		Ft. in.	Ft. in.	
20 ton—Front ...	7½	2 3	4 6	2·78
Back ...	12⅜	1 10	6 0	5·62
15 ton—Front ...	6¼	2 3	4 6	2·33
Back ...	10⅝	1 8	6 0	5·31
10 ton—Front ...	4⅞	2 0½	4 0	2·0
Back ...	7¾	1 4	5 6	4·85

TABLE IVa.

Description.	Diam. in Inches.	Width in Inches.	Axle Load in Tons.	Cwt. per sq. ft. of Area Covered.	Cwt. Max. Load per Inch Width of Tyre.
Manchester Ship Canal load diagram for highway bridges—					
Front wheel	48	15	20	5·00	13·33
Back wheel	60	18	30		16·67
Holdsworth's boiler wagon—					
Front wheel	36	9·5	15	4·85	15·80
Back wheel	42	12	25		20·83
Fowler's large ploughing engine—					
Front wheel	59	16	6	1·91	3·75
Back wheel	78	26	16		6·20
Manchester Corp. Standard load—					
Front wheel	36	10	15	4·44	15·00
Back wheel	48	12	15		12·50
Nottingham Standard loading—					
Engine—Front wheel	60·5	16	7	2·20	4·36
Back wheel	78	26	15		6·00
Trolley—Front & back wheels	42	12	16	3·81	12·33
Chester traction engine and wagons—					
Engine—Front wheel	48	8·5	6	2·31	7·06
Back wheel	84	18	10		5·55
Wagon—Front wheel	38	12	6	2·51	5·00
Back wheel	38	12	6·75		5·62
Rochester 20-ton roller—					
Front wheel	54	27	7·5	2·43	2·78
Back wheel	72	22	12·37		5·62
L.C.C. Standard tramway car—					
Front and back wheels ...	31·75	3	6·75	1·77	22·5*
Central wheels	21·75	3	3·75		12·5*
14 h.p. ploughing engine—					
Front wheel	54	16	5	1·47	3·12
Back wheel	66	20	9		4·50
Rochester 15-ton roller—					
Front wheel	54	27	6·25	2·16	2·33
Back wheel	72	20	10·62		5·31
Hove traction engine and trucks—					
Engine—Front wheel	48	10	5	1·78	5·00
Back wheel	84	22	9		4·09
Truck—Front wheel	36	9	5·5	2·62	6·11
Back wheel	45	9	6		6·67
Leicester brick truck—					
Front wheel	42	9	6	2·67	6·67
Back wheel	48	9	6		6·67
Rochester 10-ton roller—					
Front wheel	48	24·5	4·87	2·15	2·00
Back wheel	66	16	7·75		4·85
Farm wagon—					
Front wheel	36	6	2	0·83	3·33
Back wheel	48	6	2		3·33

* Running on rails.

APPENDIX II.

THE DETERMINATION OF THE BENDING MOMENTS AND SHEARING FORCES FOR BEAMS SUBJECTED TO MOVING LOADS.

By H. KEMPTON DYSON.

The determination of the bending moments and shearing forces resulting from a system of moving loads can be best made by the aid of influence lines,* and the diagrams numbered Figs. 4 and 6 have been made in this way for the three systems of loading which have been adopted as standards in the Committee's Report. The diagrams have been made for the axle loads considered as statically applied at points on one freely supported beam, and no allowance has been made for the reduction that would result from the presence of two or more girders in the width of the bridge able to take a share of the load, nor for any increase that should be made to allow for impact. Other factors that require to be taken into consideration in any practical application of the diagrams to the design of a girder bridge are :—

1. The partial or complete distribution of the concentrated loads through the filling and decking.
2. The effect of the distributed load of the decking and filling and the partial or complete loading by a crowd.
3. The presence of cross girders.
4. Fixity or continuity of the main girders.

It will be realized that Fig. 4 gives the maximum bending moment at every point of the beam for the load system moving from right to left, but as the load may come on the bridge from either direction the maximum values should be taken, namely, those shown by the right-hand halves of the diagrams for Classes 1 and 2 and the left-hand half of the diagram for Class 3. The diagrams are therefore only indicative of the effects of a rolling-load, and it is thought

* An *influence line* (Weyrauch) is a curve representing the variation of any function, such as panel load, reaction, shearing force, bending moment or stress at a particular section of a member due to unit load moving over the beam or structure. It represents the change in the function only for the section or point considered.

necessary to utter a caution against their indiscriminate use. The effects of incomplete consideration and lack of knowledge in the design of bridges are, of course, more dangerous as the span increases, but even in the case of small spans it would be as well for the bending moments and shearing forces to be calculated in detail so far as the final design is concerned, even though the diagrams may have been employed for preliminary calculations of a roughly approximate character.

Calculations are readily made when we have found the position of the system of moving loads that gives the maximum value of the bending moment or shear at the point under consideration. The following demonstration shows how the required position may be determined in each case :—

1. *Position for maximum bending moment at the point A on the beam O C* (Fig. 1B).—Replace the loads on the left of A by their resultant W_1 and the loads on the right by their resultant W_2 and draw the bending-moment influence diagram $O_1 A_1 C_1$ (Fig. 2B). Then the ordinates y_1 and y_2 directly under W_1 and W_2 represent the moments at A of unit loads placed in the position of W_1 and W_2. Thus the bending moments at A are

$$B = W_1 y_1 + W_2 y_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Let the loads move an infinitely small distance dx to the left. Then the bending moment of A becomes

$$B + dB = W_1 (y_1 - dx \tan \alpha) + W_2 (y_2 + dx \tan \beta) \quad . \quad . \quad (2)$$

Subtracting (1) from (2)

$$dB = -W_1 dx \tan \alpha + W_2 dx \tan \beta$$

and

$$\frac{dB}{dx} = -W_1 \tan \alpha + W_2 \tan \beta$$

$\frac{dB}{dx}$ is the rate of change of the bending moment and represents the shear.

Owing to the fact that with concentrated loads the shear changes by sudden steps we are unable to determine the maximum value of the bending moment by the usual expedient of equating the differential coefficient at zero, but remembering that with a point load system the shearing force diagram changes from positive to negative values at

the point of maximum bending moment we can employ the criterion that B will be a maximum when the expression $-W_1 \tan \alpha + W_2 \tan \beta$ changes sign. Now $\tan \alpha = \frac{l-x}{l}$ and $\tan \beta = \frac{x}{l}$. Therefore by substitution our criterion becomes the change of sign of

$$-W_1 \frac{(l-x)}{l} + W_2 \frac{x}{l}$$

i.e.

$$(W_1 + W_2)\frac{x}{l} - W_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

It is evident that change in value of W_1 and W_2 can only take place by one of the point loads moving over A, O or C. Now we are considering the direction of movement as being to the left. Then, if one of the point loads moves over O there is a decrease in the value of W_1, while if a load moves over C there is an increase in the value of W_2 and neither of these changes will affect the sign of expression (3); but if a load moves over A there may be increase in W_1 and decrease in W_2 which may result in change of sign. Therefore we see that the maximum bending moment will occur when a point load is placed on A and when the consideration of the point load as belonging to either W_1 or W_2 changes the sign of the foregoing expression.

Again, we see from expression (3) that we can determine the position for maximum bending moment by placing point loads in turn on A until the load on the smaller segment (including that on the point A) is greater than the total load on the whole span multiplied by the ratio which the length of the short segment bears to the whole span.

Having determined the required position of the loads the bending moment at A is readily calculated by adding up the bending moment at A of each of the loads as given by the formulæ:—

$$B_x = \frac{w}{l} d\,(l-x) \text{ for a load } w \text{ situated to left of point A,}$$

and $B_x = \frac{w}{l} x\,(l-d)$ for a load w situated to right of point A,

where d is the *distance* that the load is situated from the left-hand end of the beam.

2. *Position for maximum shearing force at the point A on the beam OC* (Fig. 1B).—The shear influence line

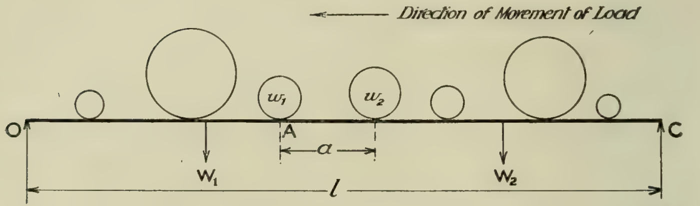


FIG. 1B

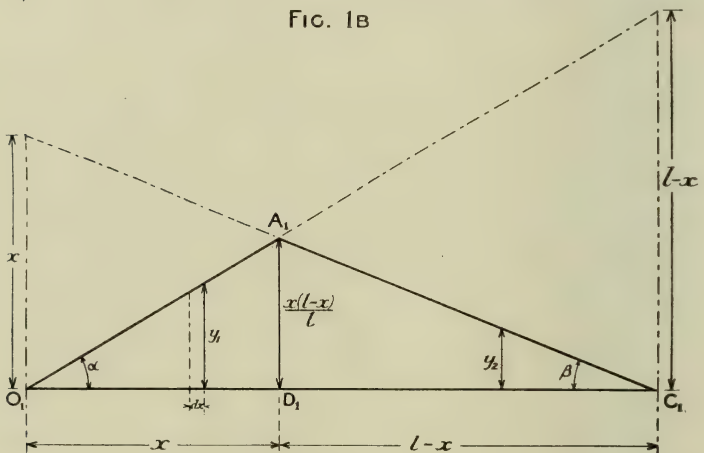


FIG. 2B

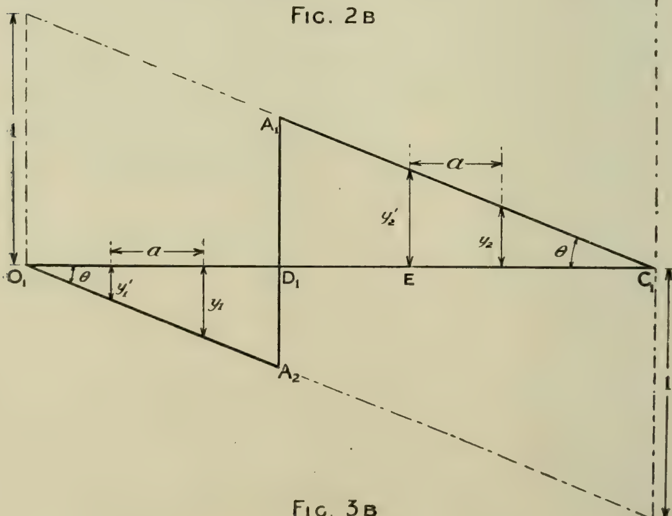


FIG. 3B

diagram is shown in Fig. 3B. If a load be moved to the left it is evident that it will increase the negative shear until the load passes the point A; consequently the shear at A is a maximum when one of the loads is situated immediately over it. It is again evident that while the loads to the right of A gradually increase the total shearing force when gradually moved towards the left, those to the left decrease it. Generally the maximum shear at A occurs with few or no loads on the smaller segment.

Now let the distance apart of two consecutive loads w_1 and w_2 be a, and let them be brought in turn to the point A. Of the total load W call the resultant of the load to the left W_1 and the resultant to the right W_2, as before. Then directly w_1 passes A the shear is decreased by an amount w_1, thus becoming $W_2y_2 - W_1y_1$. If the load system continues to move to the left the shearing force is gradually increased until w_2 is infinitely close to A when the shear becomes $W_2y_2' - W_1y_1'$

Now

$$\frac{y_2'}{C_1 E} = \frac{y_2}{C_1 E - a} = \tan\theta = 1/l$$

Therefore

$$y_2' - y_2 = C_1 E \tan\theta - (C_1 E - a)\tan\theta = a\tan\theta$$

Similarly

$$y_1 - y_1' = a\tan\theta$$

The increase in shear is

$$\begin{aligned} W_2y_2' - W_1y_1' - (W_2y_2 - W_1y_1) \\ &= W_2(y_2' - y_2) + W_1(y_1 - y_1') \\ &= W_2 a\tan\theta + W_1 a\tan\theta \\ &= (W_1 + W_2)a\tan\theta = W a\tan\theta = \frac{Wa}{l} \end{aligned}$$

Hence either w_1 or w_2 when placed at the point A will cause the shearing force to be a maximum according to whether w_1 is greater than or lesser than Wa/l, that is, according as $\frac{w_1}{a} \gtrless \frac{W}{l}$. Thus, to determine the position of the load which will give the maximum shear at any point, work out the value of W/l (i.e. the average load for the whole span) and then values of w/a (i.e. the average load for the distance

between any two contiguous loads). The individual load which affords the greatest difference of the latter value from the former will, when placed on the point, cause the greatest shear at the point.

The foregoing rules for determining positions for maximum bending moment and shear at a point A apply only so long as one of the individual concentrated loads does not, by reason of the movement, roll off the span. If it does, several trials may be necessary to find the proper position.

Calculations for the effects of rolling loads on continuous beams are tedious unless assistance is made of tables of lengths of ordinates of influence line diagrams determined for two or more spans bearing various ratios one to another, such ordinates being expressed as ratios of the value obtained by multiplying the load by the length of the span, as for example those given in Gustav Griot's "Interpolierbare Tabellen zum raschen Auftragen der Einflusslinien für Momente und Scheerkräfte sowie der Kurven für verteilte Lasten" (Zürich : Schulthess & Co.).

Attention is called to the fact that with continuous beams, especially where short spans adjoin long ones, the imposition of super-loads on one bay when the adjoining bay only receives the dead load, frequently causes negative bending moment throughout the latter. This may mean in practice that supports hold down the ends of beams and prevent continuity extending beyond the supports of the adjoining span, or that the beam may be lifted off one or more supports. Continuous construction requires very careful consideration in order to secure proper stability with economy.

SIXTY-SIXTH ORDINARY GENERAL MEETING

WEDNESDAY, FEBRUARY 16, 1916

THE SIXTY-SIXTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held at Denison House, 296, Vauxhall Bridge Road, Westminster, London, S.W., on Wednesday, February 16, 1916, at 5.30 p.m.

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., M.S.A., F.S.I., etc.), in the Chair.

MR. CHARLES F. MARSH, M.Inst.C.E., M.I.Mech.E., M.Am.Soc.C.E., V.P.C.I., read the following paper :—

REINFORCED CONCRETE AS APPLIED TO WATERWORKS CONSTRUCTION

IN the construction of works specially required for the purpose of the conveyance and storage of water reinforced concrete can be usefully employed for many purposes.

The chief uses for this material are the construction of dams for impounding water, of aqueducts and pipes for its conveyance, and reservoirs and tanks for its storage.

There are also many works of an incidental nature for which reinforced concrete may be employed, amongst which are Retaining Walls, Bridges, Slope Coverings, Buildings, Fencing, and sundry other structures of various descriptions essential for the completion of any large system of water supply.

In this paper only the main uses of the material will be dealt with, as the incidental uses entail its general application to engineering works, and not its special use in waterworks construction.

CONCRETE, ETC.

The concrete used for structures, which must resist the pressure of water, should be richer in cement than that used for the generality of structures.

For reservoirs, tanks, and dams, where there is sufficient thickness, the concrete should be mixed in the proportions of 1 : 1½ : 3, or 810 lbs. of cement to 13½ cub. ft. of sand and 27 cub. ft. of broken stone or shingle, which mixture is suffi-

ciently watertight for any but very considerable heads, but for pipes and structures of small thickness, say less than 3 in., a mortar mixed in the proportions of 1 to $1\frac{1}{2}$, or 1,620 lbs. of cement to 27 cub. ft. of sand, should be used. This mixture is, of course, no more resistant to water pressure than a $1 : 1\frac{1}{2} : 3$ concrete, but in a thin structure there is a danger in the use of stone or shingle, since two pieces may possibly come together, and any failure in the proper consolidation of the concrete may leave a plane of leakage through the concrete. The size of the broken stone or shingle should not exceed such as will pass through a $\frac{3}{4}$-in. square-meshed sieve, and may with advantage be $\frac{1}{2}$-in. gauge. It is not advisable to use a richer mixture, as rich mixtures shrink more when drying and expand more when wet than leaner mixtures, and consequently cracks are more likely to be induced ; while it has been proved conclusively that with proper care in mixing and placing, a mixture in the proportions of 1 to $1\frac{1}{2}$ to 3 is practically impervious under considerable heads.

For pipes under pressures exceeding about 40 ft. special linings should be used, such as the sheet steel tubing in a Bonna pipe or other suitable layer of impervious material.

For any structure which has to resist the percolation of liquids it is advisable, in my opinion, to mix some waterproofing compound with the concrete, or otherwise to provide against leakage by the use of a soap and alum wash, paraffin wax, or other suitable protective coating.

For increasing the imperviousness of concrete or mortar ordinary hydrated lime has been used very successfully and appears just as effective as any of the patent compounds on the market. It can be used in proportions up to 10 per cent. of the weight of cement without injuriously affecting the strength of the mixture, but 5 per cent. is sufficient for any ordinary purpose.

The concrete should always be kept damp for some time after moulding, depending on the richness of the mixture. The period in the case of a $1 : 1\frac{1}{2} : 3$ mixture should be about four weeks.

In structures of considerable length, which may be alternately wet and dry and are exposed to the variations of temperature, it is advisable to provide against cracking which is almost certain to occur. Such structure should have specially constructed contraction joints not more than 30 ft. apart, leakage being prevented at the joints by the insertion of sheet lead or copper baffles extending well into the

concrete on each side of the joint and bent over at the extremities to form a good key.

In the construction of all structures to resist the pressure of liquids, special care is necessary to provide adequate reinforcement against shrinkage due to the setting of the concrete, fall of temperature, and excessive dryness.

Dams.

Dams for impounding water have not been constructed of reinforced concrete in this country to any extent up to the present time, but there have been many cases in America where this material has been used for such dams.

When reinforced concrete is employed for the construction of dams they are usually of a hollow form of construction, having up-stream and down-stream slabs supported on cross walls carried upon a foundation slab.

From the foundation slab a core wall must be carried down under the bottom of the up-stream slab, and extended well into a watertight stratum. The provision of an adequate cut-off wall is most essential, as some of the dams already constructed of this material have failed owing to the neglect of this precaution.

The up-stream slab is generally constructed with a flat slope not steeper than 1 to 1, since the flatter the slope the more uniform is the pressure on the base.

With a dam of this type, as the reservoir fills, the line of resultant pressure on the base will at first become farther and farther up stream from the centre of pressure on the base with the reservoir empty. When the reservoir has filled for a certain proportion of its depth depending on the slope of the up-stream slab, the line of resultant pressure on the base moves back towards the centre until under the limiting flood conditions it will generally be found to be slightly on the downstream side of the centre of pressure on the base with the reservoir empty.

The resultant pressure will in no case move very far from the centre of the base, and consequently the intensity of pressure will never vary greatly between the up-stream and down-stream extremities of the base.

In the case of a solid masonry dam the centre of pressure on the base with the reservoir empty is usually at the upstream extremity of the middle third of the width, and with reservoir under maximum flood is usually at the down-stream limit of the middle third of the width. The consequence

of these limits is that with reservoir empty there will be no pressure at the down-stream toe and a maximum pressure at the up-stream heel, while with reservoir full there will be no pressure at the heel with a maximum pressure at the toe. As the reservoir fills the pressure intensity will vary between these extremes.

A further advantage of the hollow type of dam is that the structure is very much lighter, with a consequent reduction of the pressure on the base, whereas with a solid masonry dam the weight of the dam when over a limiting height,

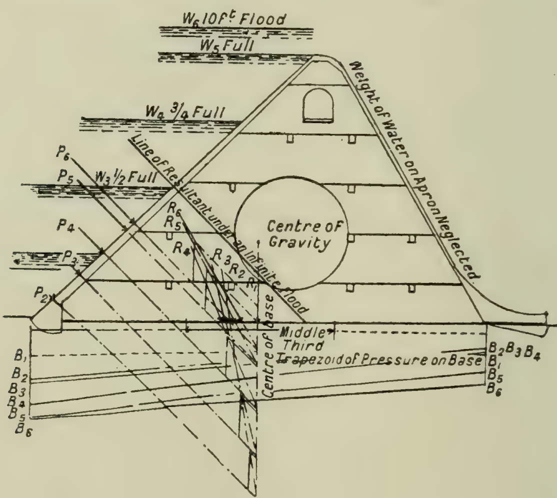


FIG. 1.

depending on the specific gravity of the materials of which it is composed, necessitates an increased widening of the lower portion of the structure in order to keep the pressure within safe limits.

Fig. 1 shows a hollow dam 108 feet in height from the foundation to the crest and with an up-stream slab with a slope of 1 to 1. The greatest variation in the intensity of pressures on the base is shown by the line B_4 B_4 when the reservoir is $\frac{3}{4}$ full, B_1 B_1 indicates the variation in the pressure intensities when the reservoir is empty, B_2 B_2 when $\frac{1}{4}$ full, B_3 B_3 when $\frac{1}{2}$ full, B_5 B_5 when full, and B_6 B_6 when 10 ft. of water is passing over the crest.

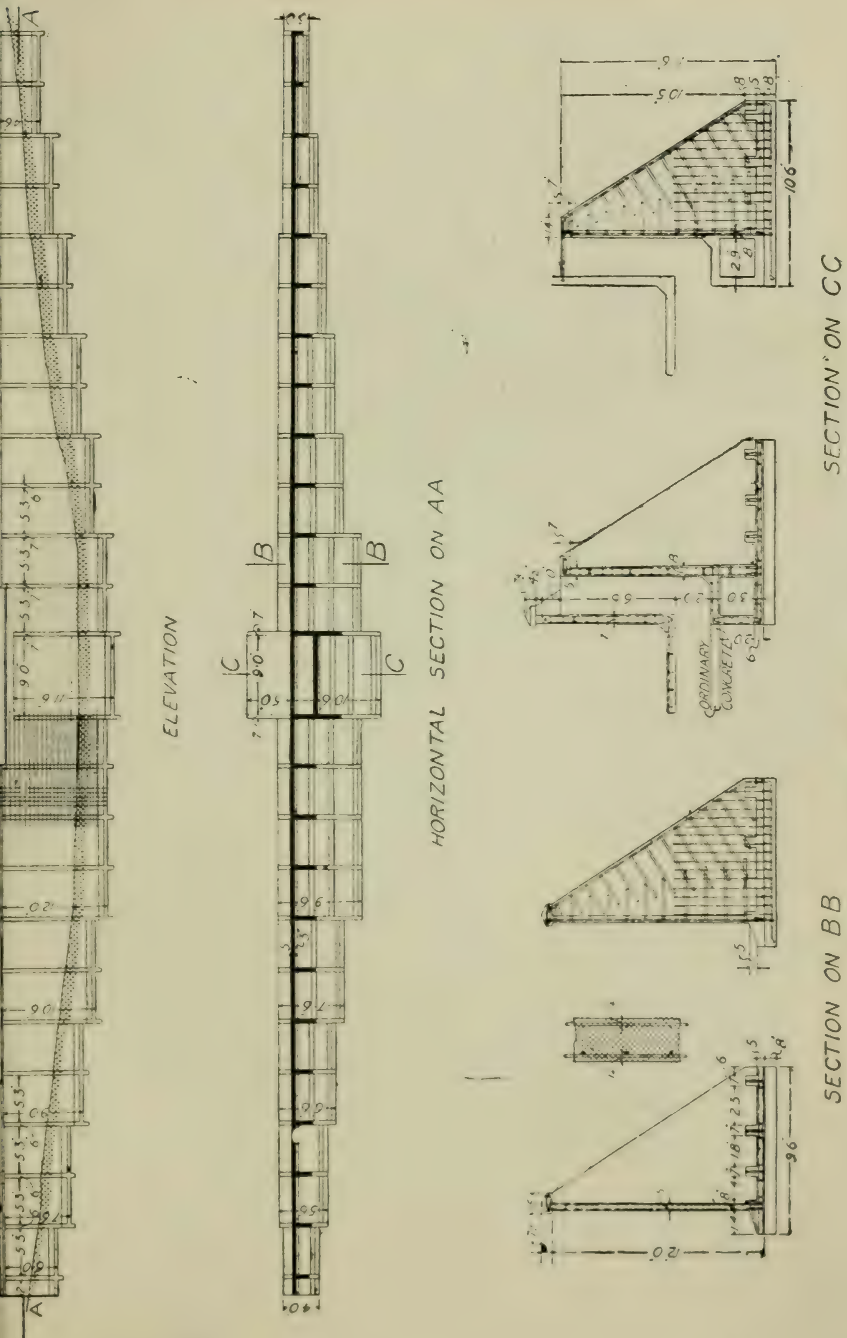


Fig. 2.—Shipley Glen Dam: Details of Construction.

The base slab of a hollow dam is generally formed with holes to allow any water which may gain access under the dam free vent and so obviate any tendency to uplift. The water percolating through the base slab is discharged under the bottom of the down-stream slab to the stream below the dam.

The crest of the dam is formed of such shape as will accommodate the flow of the greatest height of flood which will be allowed to pass over without the formation of a vacuum under the overfall. The down-stream slab is generally constructed of a considerably flatter slope than that usual for solid masonry dams, and the water is consequently conducted to the stream below the dam with less velocity and with a more even flow. This slab frequently terminates before reaching the base of the dam, allowing free access for the water to and from the interior, or sufficient holes are left at its base to effect the same object.

The cross walls or buttresses are stiffened by longitudinal beams, and may be formed with openings to economize material and give means of access.

There is usually a passage-way through the dam from end to end, and frequently several at various depths. The gearing for the outlet and scour valves may be actuated from these passage-ways.

Dams, when of small height, are sometimes constructed without a down-stream slab. Fig. 2 shows such a dam constructed by the Yorkshire Hennebique Contracting Company at Shipley Glen ; it has a length of 146 ft. and a maximum height of 14 ft. 6 in.

Elevated Tanks.

Perhaps one of the most economical uses of reinforced concrete is in the construction of elevated tanks, of which there are many examples in existence. With care and skill in the design these structures may be made quite picturesque features of the landscape, and are in any case less objectionable than elevated steel tanks on framed supports. A reinforced concrete tank can be constructed at a cost of from 40 per cent. to 50 per cent. that of a tank formed of riveted steel plates, and they will in general be less expensive than tanks of pressed steel or cast-iron plates.

When designing circular reinforced concrete tanks it is, in my opinion, advisable to limit the working resistance of the steel to 12,000 lbs. per sq. in., since, although the tensile resistance of the concrete is neglected, the elongation

of the steel bars must induce elongation in the concrete, and a higher stress in the steel will in all probability cause the

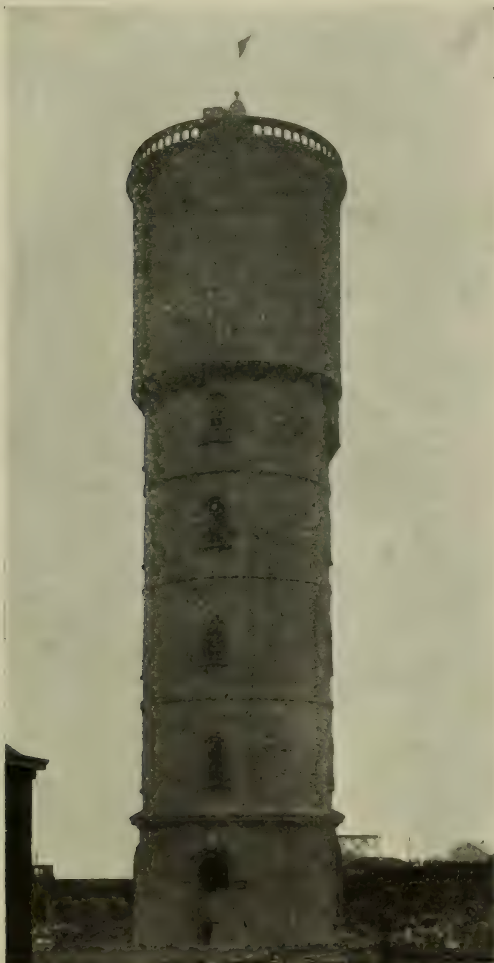


FIG. 3.—Water Tower at Cleethorpes.

concrete to crack. It is also, in my opinion, advisable to insert two series of circular rings in the walls of the tank, one near each surface, in place of one series at the centre of the

thickness. One case has occurred to my knowledge, and there may have been others, where the concrete outside the central series of hoops broke away and caused the failure

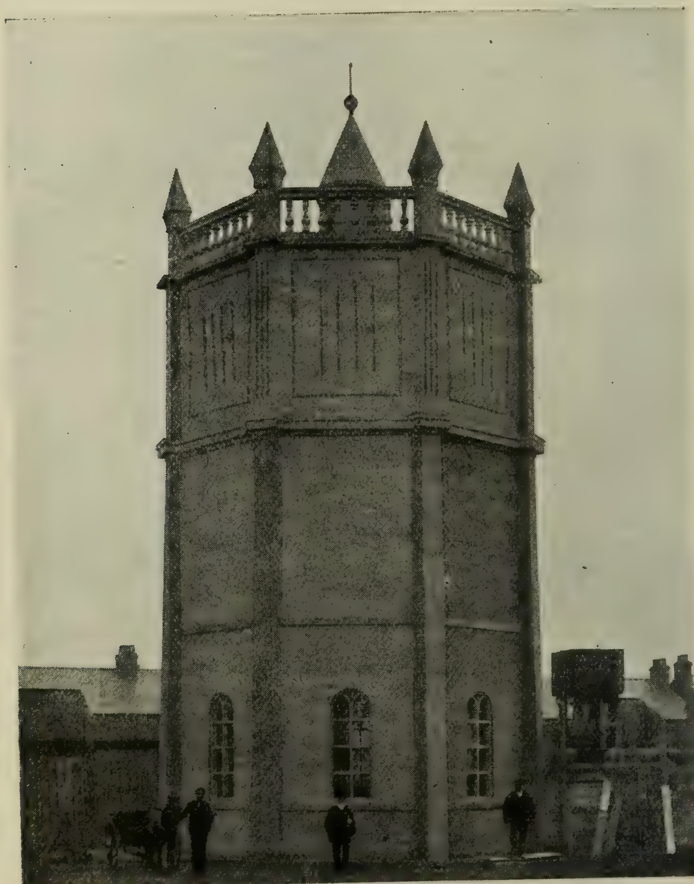


FIG. 4.—Water Tower at Immingham.

of the structure. It must be remembered that whereas the interior of the tank is kept at a fairly constant state of moisture and temperature the outside is exposed to the variations in temperature and humidity, and consequently the two surfaces

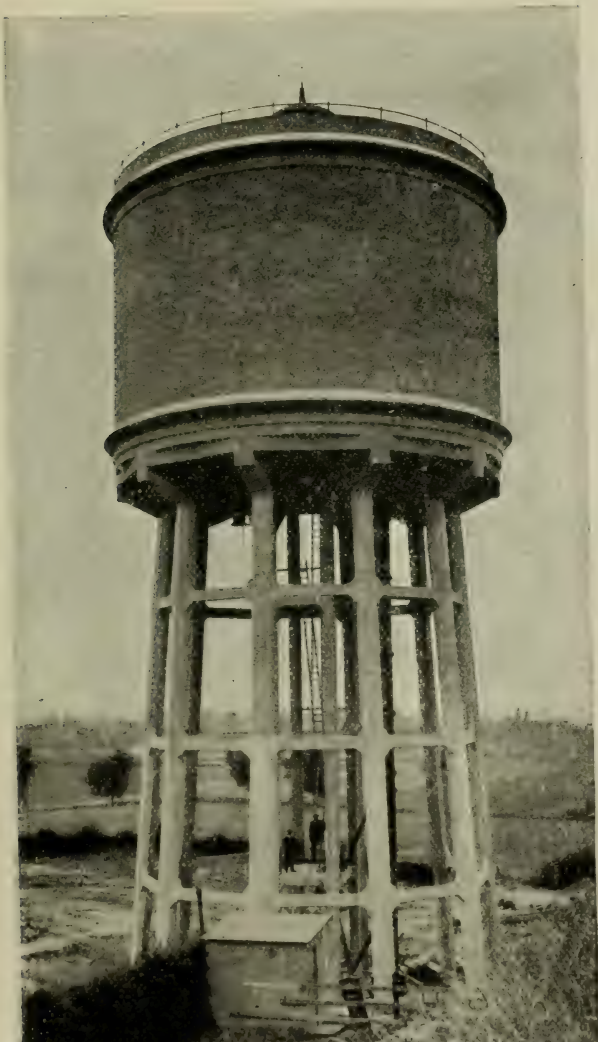


FIG. 5.—Water Tower at York.

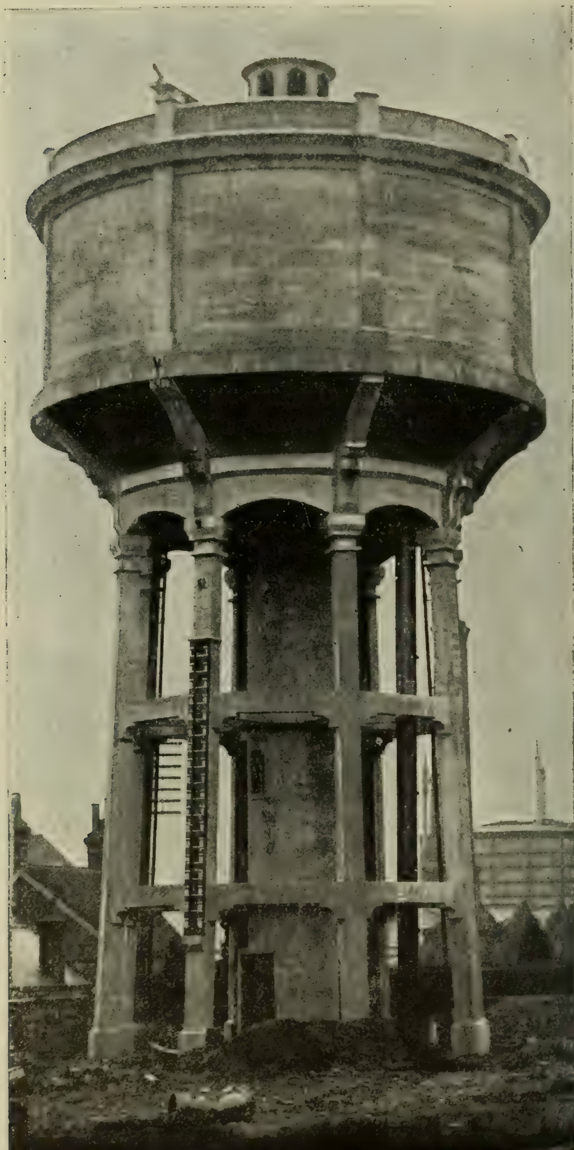


FIG. 6.—Water Tower for the St. Albans Water Company.

are under very different conditions affecting the expansion and contraction of the concrete.

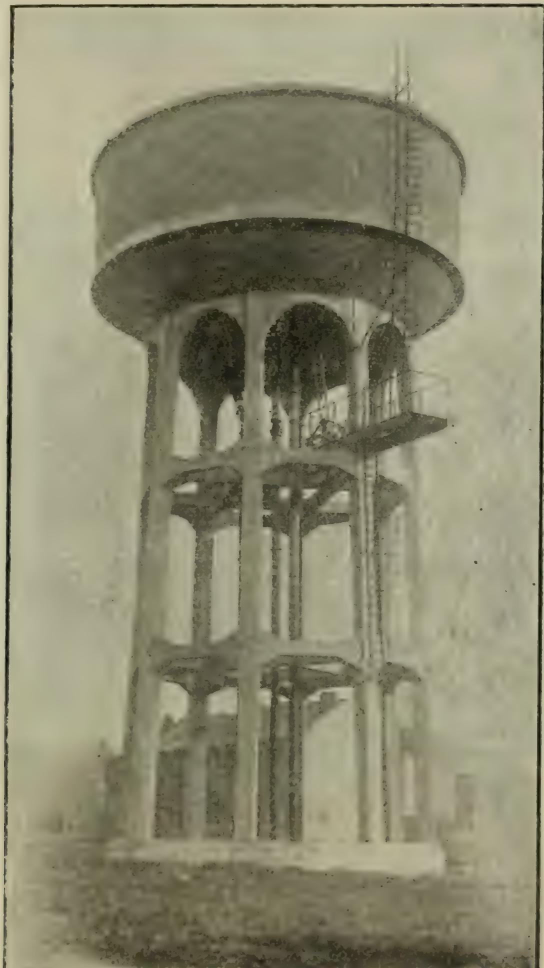


FIG. 7.—Water Tower at Portsmouth.

Most of us have seen examples of elevated tanks, but it may be interesting to describe a few typical examples.

Fig. 3 is a view of Cleethorpes water tower, which is believed to be the highest tank in this country. It was constructed by the Indented Bar and Concrete Engineering Company, and is 174 ft. above ground-level, and has a capacity of 400,000 gallons. The inside diameter is 34 ft. and its depth 42 ft. The tank is of steel and is surrounded by a reinforced concrete mantel wall 3 in. thick, the space between this wall and the outside of the tank being 2 ft. 6 in. The tower supporting the tank is 18 in. thick at the base and 12 in. at the underside of the tank.

Fig. 4 shows the water tower at Immingham, constructed by the same company for the Great Grimsby Waterworks Company. It is given as representing the artistic effect which

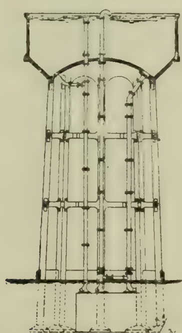


FIG. 8.—Water Tower at Portsmouth.

may be achieved in the design of such structures. It is 45 ft. high, 26 ft. in extreme width, the depth of the tank is 12 ft. 6 in., and its capacity 40,000 gallons. In the centre of the tank there is a hollow octagonal column 6 in. thick supporting the roof and containing a stairway for obtaining access to the top of the tank. The roof slab is 3 in. thick, supported on beams converging from the angles of the tank walls to the angles of the interior hollow column.

Provision has been made to heighten the tower by 40 ft. at some future date, and to construct an additional tank of 60,000 gallons' capacity.

Fig. 5 is a water tower constructed on the Kahn system for the York Waterworks. It has a capacity of 300,000 gallons and a total height of 118 ft. 6 in., the depth of the

tank is 30 ft., and the internal diameter 46 ft. 6 in. The thickness of the circular walls is 9 in. at the bottom, reducing to 6 in. at the top, and the thickness of the bottom is 12 in. A circular shaft 6 ft. in diameter is constructed through the centre of the tank and provides access to the top, which is covered by a dome and lantern. The tank is supported on 14 columns, 4 in the centre and 10 on the outside.

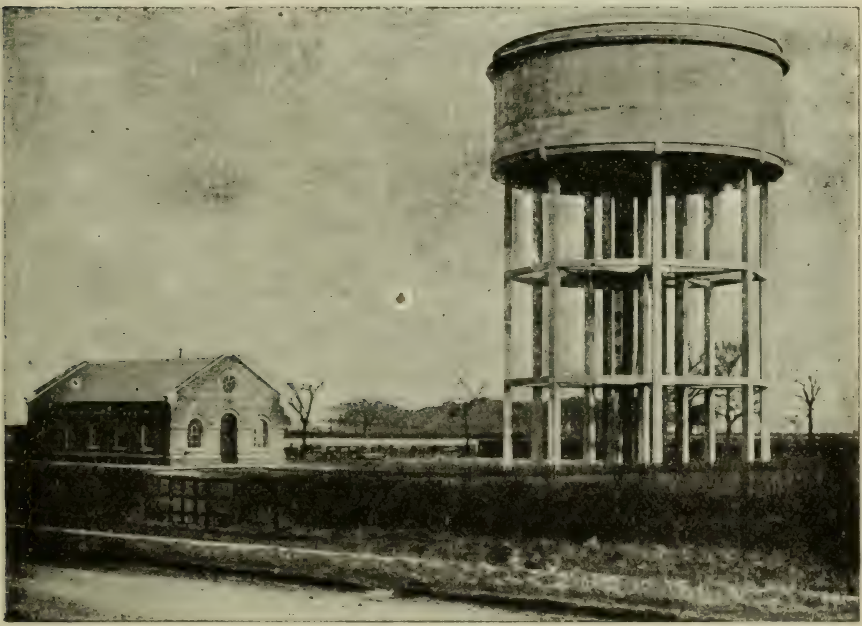


FIG. 9.—Pumping Station and Water Tower at Hatfield, near Doncaster.

Fig. 6 is a water tower constructed by the same firm for the St. Albans Waterworks Company, with a capacity of 80,000 gallons. It is of the overhung type, in which the bottom of the tank is cantilevered out, producing a very pleasing feature.

Fig. 7 shows a water tower designed by Messrs. Mouchel Partners and constructed by the Yorkshire Hennebique Contracting Company for the Borough of Portsmouth salt-water system. It has a capacity of 60,000 gallons and is 48 ft. 3 in. high. The internal diameter of the tank is 32 ft., and the

depth varies from 10 ft. to 14 ft. 6 in. Fig. 8 is a sectional view showing the form of construction. The average thickness of the tank walls is $4\frac{1}{2}$ in., and that of the floor $5\frac{1}{2}$ in.

Fig. 9 is an uncovered water tower designed and constructed by the same firms for the Thorne and District Waterworks at Hatfield, near Doncaster, with a capacity of 150,000 gallons. The bottom of the tank is 50 ft. above ground-level. The internal diameter 45 ft., and the depth of water 15 ft. The

FIG. 10.—Water Tower at Rolleston, near Burton-on-Trent.

average thickness of the tank walls is 5 in., and that of the floor $5\frac{1}{2}$ in.

Fig. 10 shows a water tower at Rolleston, near Burton-on-Trent, with a capacity of 45,000 gallons, constructed by Messrs. Coignet & Co. The structure is about 50 ft. high, and has two inside suspended floors and a staircase. The thickness of the walls of the tank is 4 in., and that of the bottom 5 in. The dome has a thickness of 3 in. The depth of water in the tank is about 15 ft., and there is a cylindrical passage through the tank to give access to the roof.

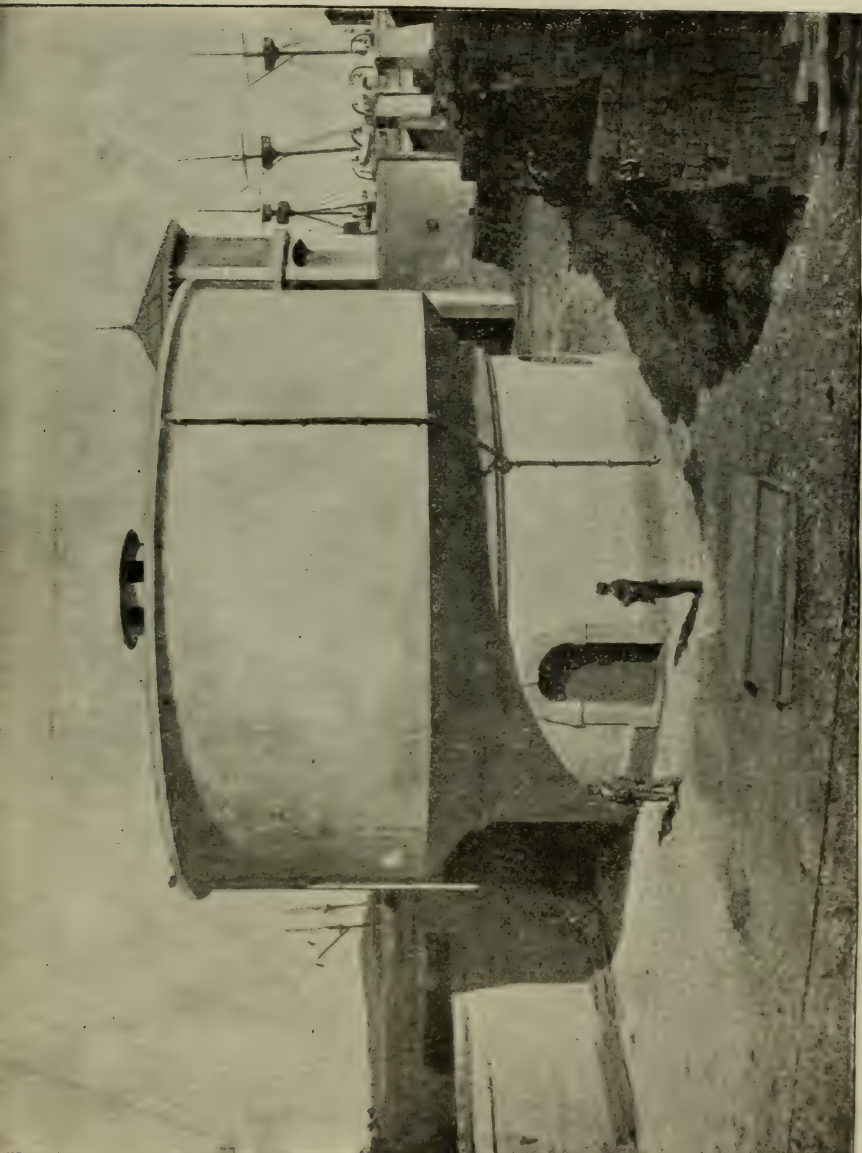


Fig. 11.—Water Tank at Toulon Arsenal.

Fig. 11 shows one of two elevated tanks with a capacity of 110,000 gallons, constructed by Messrs. Coignet for the Toulon Arsenal, France, some sixteen years ago. The bottom is constructed on a circular masonry wall, and is dome-shaped, the outer walls being supported by the cantilevered portion of the bottom, as shewn. This was one of the first tanks to be constructed in this manner.

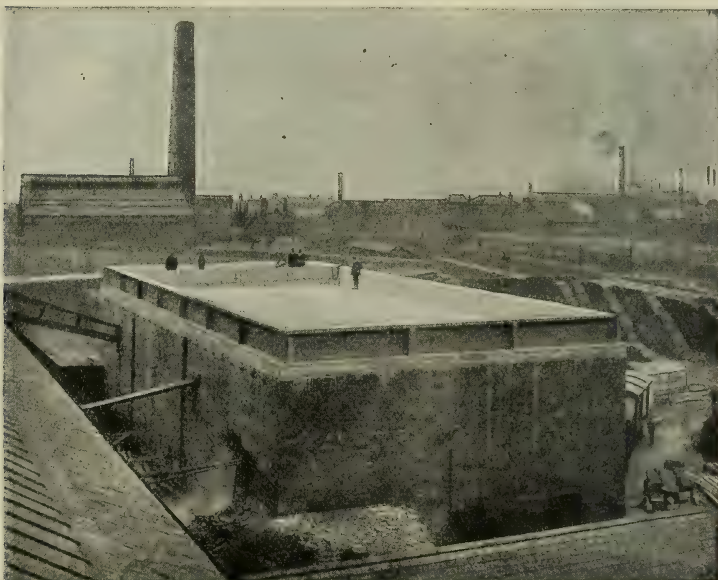


FIG. 12.—Tank for West Hartlepool Paper Pulp Company.

The thickness of the walls is 4 in., and that for the bottom 5 in. There is a cylindrical passage through the centre of the tank to obtain access to the roof.

Fig. 12 shows a tank constructed by Messrs. Coignet on the top of a boiler-house for the West Hartlepool Paper Pulp Company. The depth of water is 4 ft. The thickness of the walls is 5 in. and their height 5 ft.

The mixture of concrete used for tanks by Messrs. Coignet is usually in the proportion of 1 to 2 to 4, the inside of the tanks being rendered with sand and cement to a thickness of $\frac{1}{2}$ in. for the purpose of securing water-tightness.

In the construction of tanks it is sometimes the practice

to render the inside with sand and cement to which is added a waterproofing compound. Some constructors rely entirely on the imperviousness of the concrete when properly graded, proportioned, mixed, and deposited ; others coat the inner surface with some retentive composition such as paraffin wax, while others mix a waterproofing substance in the concrete used in the construction. The proportion for the concrete most generally adopted for the tank itself is about 1 to $1\frac{1}{2}$ to 3, while a mixture in the proportion of about 1 to 2 to 4 is almost universally used for the supporting columns, bracing, and beams, and also for the roof.

Reservoirs Entirely or Partly in the Ground.

In the construction of reservoirs of this nature the question as to the economy or otherwise resulting from the use of reinforced concrete should receive careful consideration before its employment is decided upon. The method of design, if this material is decided upon, will also require careful consideration.

In many cases it is not economical, in my opinion, to use reinforced concrete for the walls or floor, but it is almost universally an economical material for roof construction.

If a considerable portion of the depth is below ground the form of retaining wall construction, with a bottom slab at the back tied to the front slab by ribs, is not an economical form of construction, as the excavations have to be considerably enlarged to accommodate the bottom slab.

A wall designed as a cantilever, supported from the floor of the reservoir, will reduce the excavation, but great care is necessary to provide ample support at the bottom to prevent failure between the base of the wall and the floor when the reservoir is empty.

If the reservoir is covered and the covering can be constructed before an excessive loading is brought upon the walls, the roof beams and similar beams formed in the floor can be constructed to support beams along the top and bottom of the wall, which in their turn support the ends of vertical beams between which the walls of the reservoir can be constructed as slabs with horizontal reinforcements.

The covering usually adopted for reservoirs is of the beam and slab type, similar to ordinary floors, and supported by columns, but small circular reservoirs may be covered with a flat dome in a similar manner to that frequently employed for elevated tanks.

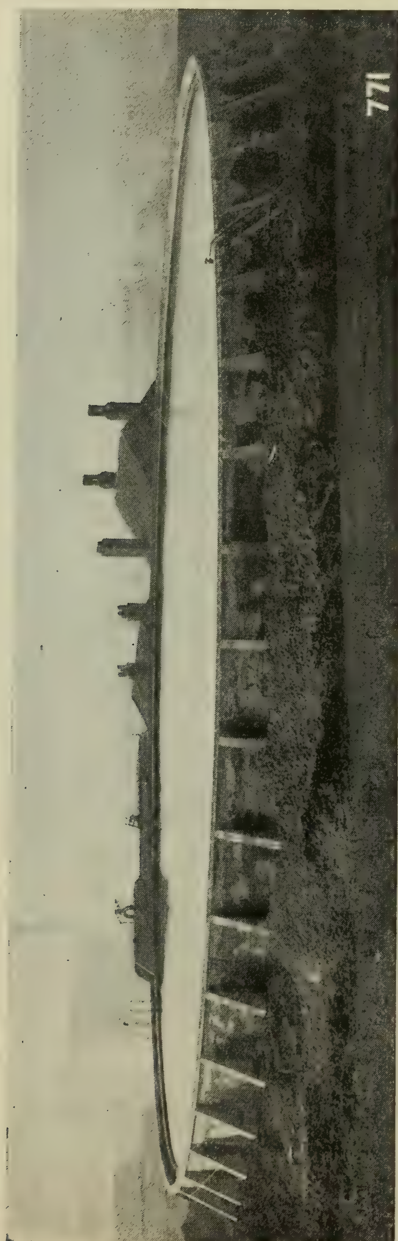


Fig. 13.—Reservoir at Runcorn.

Fig. 13 shows a large circular uncovered reservoir for water-cooling purposes, with a capacity of 2 million gallons, constructed by the Indented Bar and Concrete Engineering Company at Runcorn for the Castner Kellner Alkali Company. The reservoir is almost entirely above ground-level, and consequently the type of construction used was an economical one. The radius was so large that the resistance

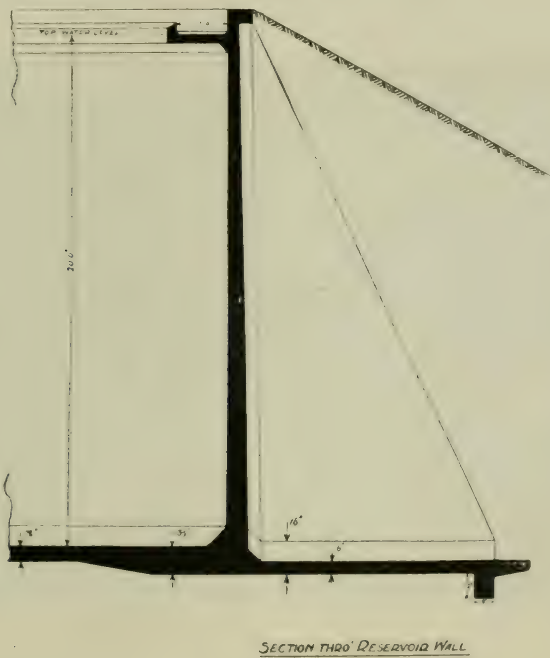


FIG. 14.—Reservoir at Runcorn.

as a cylinder was not relied upon. The internal diameter is 143 ft. and the depth 21 ft. The wall is 11 in. thick at the base, decreasing to 5 in. at the top, and is supported by buttresses 10 ft. apart, centre to centre, 10½ in. thick and 9 ft. 9 in. long at the bottom, reducing to nil at the top. The base slab projects 11 ft. behind the wall and is 6 in. thick. The horizontal beam near the extremity of the slab is 12 in. deep by 9 in. wide. The floor of the tank is 7½ in. thick,

except near the wall, where it increases to 13½ in. in thickness as it here forms part of the base of the wall.

Fig. 14 shows a section of the wall. The trough at the top is formed for cooling the water, which is pumped in warm at the top and drawn off cold from the bottom.

The walls were designed to resist the internal pressure without any support from the earth backing, and were absolutely watertight without any special treatment beyond careful selection of ingredients for the concrete, correct proportioning, and proper mixing.

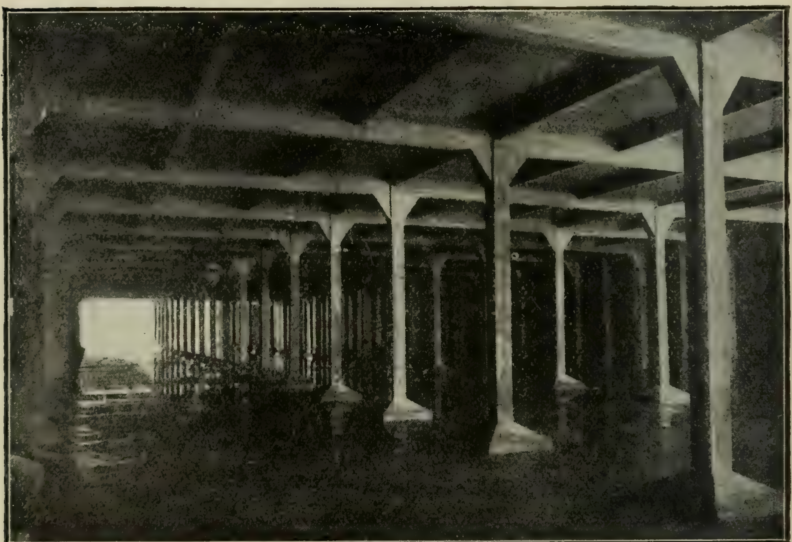


Fig. 15.—Interior View of Completed Reservoir.

Fig. 15 is a view of the interior of a covered reservoir for Cowes, designed by Messrs. Mouchel Partners, and shows a reinforced concrete roof of the usual type.

Aqueducts.

Reinforced concrete is a suitable material for elevated aqueducts, a notable example being the aqueduct used to convey water from the Rhone to the Simplon Tunnel works which was designed and constructed by M. Hennebique, and

is illustrated in "Reinforced Concrete" by Marsh and Dunn. This aqueduct was nearly 2 miles long with a fall of 1 in 830. It was of box section about 6 ft. 3 in. by 6 ft. 3 in., and the side walls were about 4 in. thick, the floor about 4 in. at the sides, increasing to about 6 in. at the centre, and the roof about $3\frac{1}{8}$ in. at the sides and $4\frac{3}{4}$ in. at the centre. The aqueduct was supported on reinforced concrete columns varying from 13 to $19\frac{1}{2}$ ft. in height and placed about 16 ft. 6 in. centre to centre. Expansion joints were left over each support, with a sheet of flexible metal embedded near the outside. These joints were filled in after the first contraction of the concrete.

Open aqueducts built almost entirely above ground-level and those for carrying water over valleys may, with economy, be constructed of reinforced concrete, but for those constructed mainly below ground-level this material will not be so economical for the same reasons as given in the case of reservoirs.

Pipes.

Pipes under small heads, say up to about 40 ft., may be constructed of reinforced concrete without any special impervious material being embedded in the thickness. Fig. 16 shows the design for such a pipe 7 ft. internal diameter under a head of $38\frac{1}{2}$ ft.

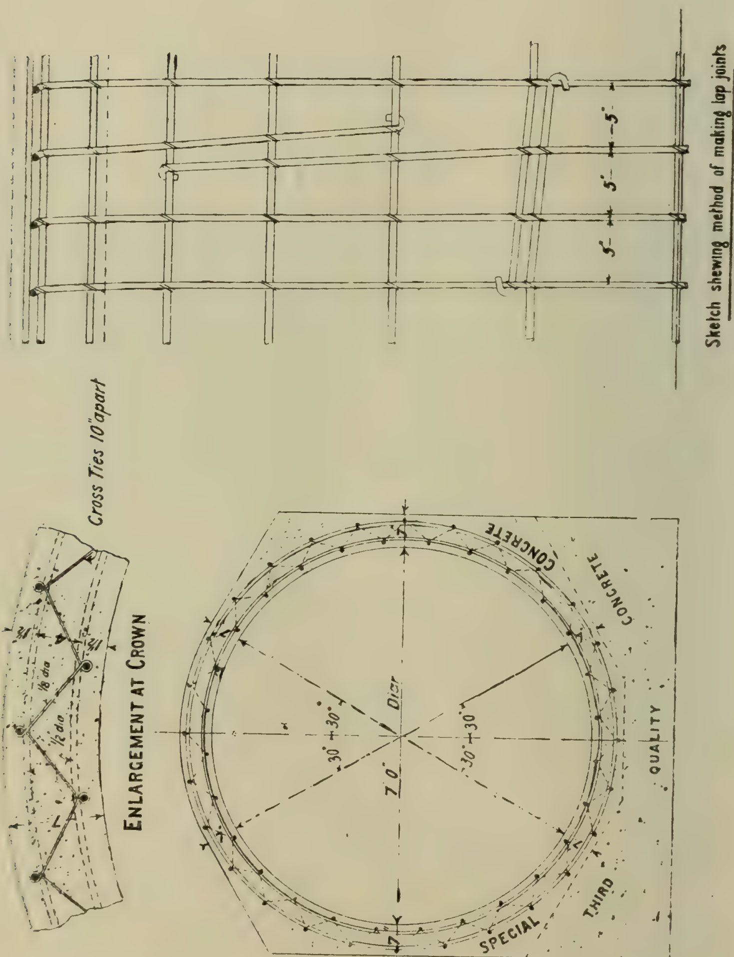
For heads over about 40 ft. some special impervious layer is, in my opinion, necessary. The well-known Bonna pipe is a good example of this type of construction. The impervious layer is formed by a thin sheet steel tube with longitudinal welds formed by the oxy-acytelene or electric processes. This tube is embedded in the thickness of the concrete with the reinforcement outside for smaller heads and inside and outside for greater heads.

Fig. 17 shows the process of welding the steel tubes of pipes 1·5 metres (4·92 ft.) diameter for Paris Waterworks. The length of this main was nearly 3 miles and the maximum head about 213 ft.

Fig. 18 shows two complicated tubes for the same works, the larger being a four-way branch, the main pipe being 1·25 metres (4·11 ft.) diameter and the branches each 0·6 metre (2 ft.) diameter.

Fig. 19 shows the winding of the spiral reinforcements and the attachment of the longitudinals.

Fig. 20 shows the tube and reinforcement for an **S** bend 1·1 metres (3·6 ft.) diameter.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5'
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FIG. 16.

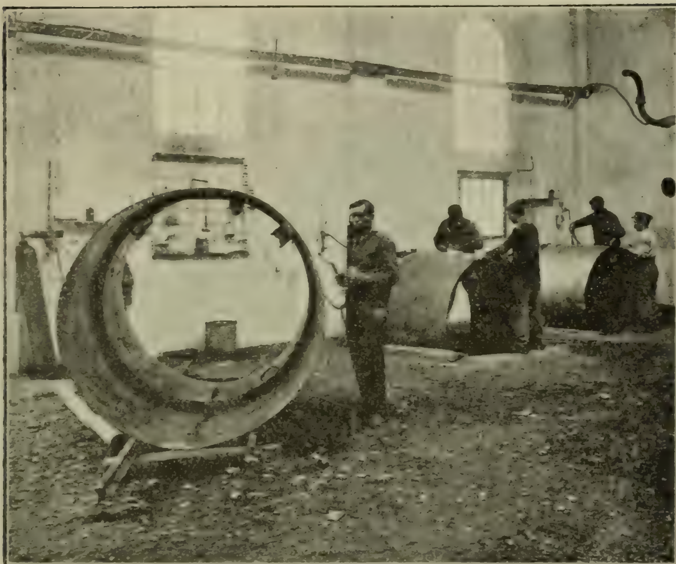


FIG. 17.

FIG. 18.

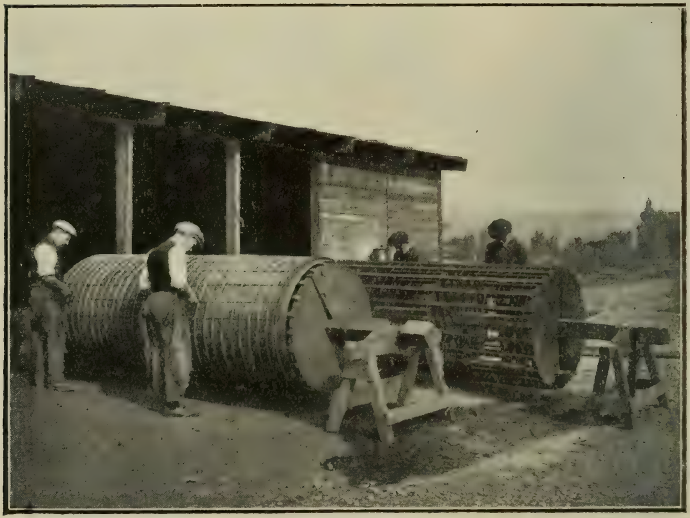


Fig. 19.

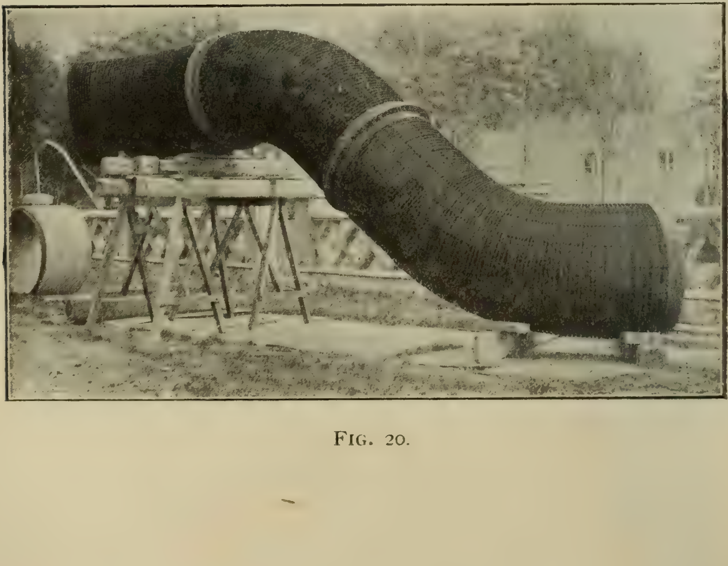


Fig. 20.

Fig. 21 shows the tube and reinforcement for a breeches piece 1·8 metres (5·9 ft.) diameter.

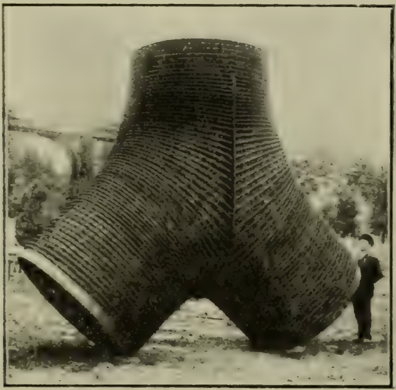


FIG. 21.

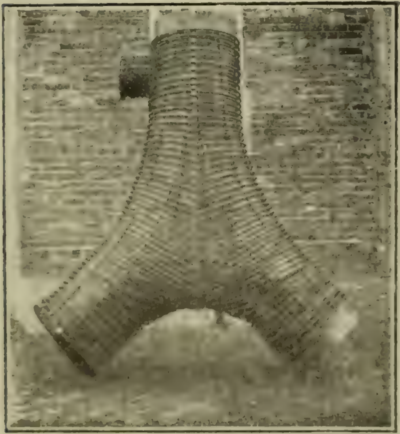


FIG. 22.

Fig. 22 shows a breeches piece with a small branch.

Fig. 23 shows a pipe 1·8 metres (5·9 ft.) diameter with a curved branch 1·5 metres (4·92 ft.) diameter.

Fig. 24 shows a taper pipe with horizontal invert.
Fig. 25 shows a venturi meter tube on a pipe 1·25 metres

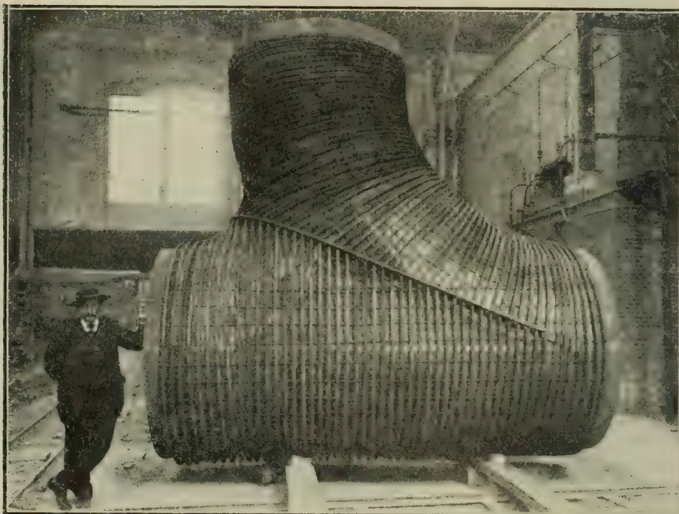


FIG. 23.

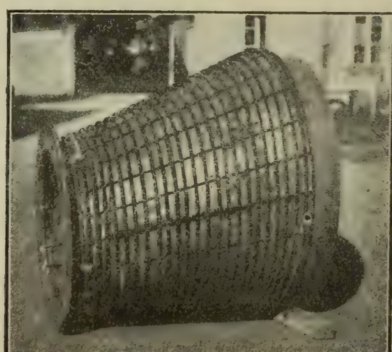


FIG. 24.

(4·1 ft.) diameter, and Fig. 26 is a pressure ring piece for this metre.

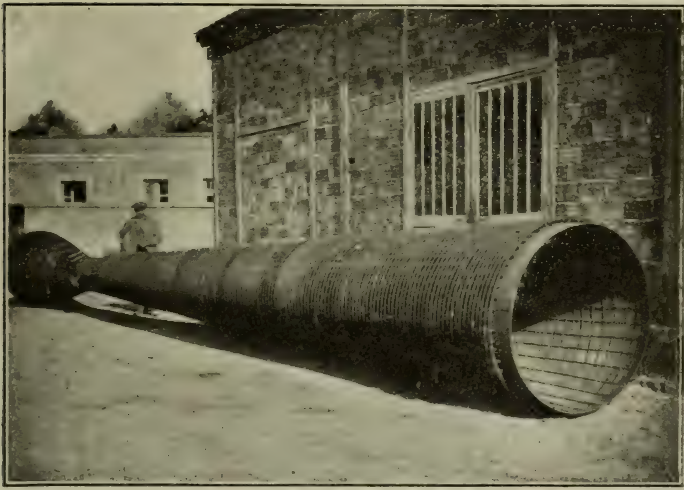


FIG. 25.

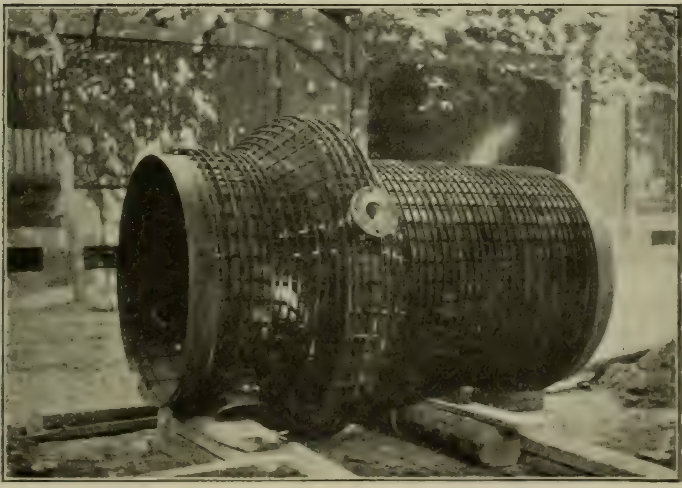


FIG. 26.

Reinforced concrete pipes when of convenient diameter are moulded vertically, the reinforcement and the steel tube,

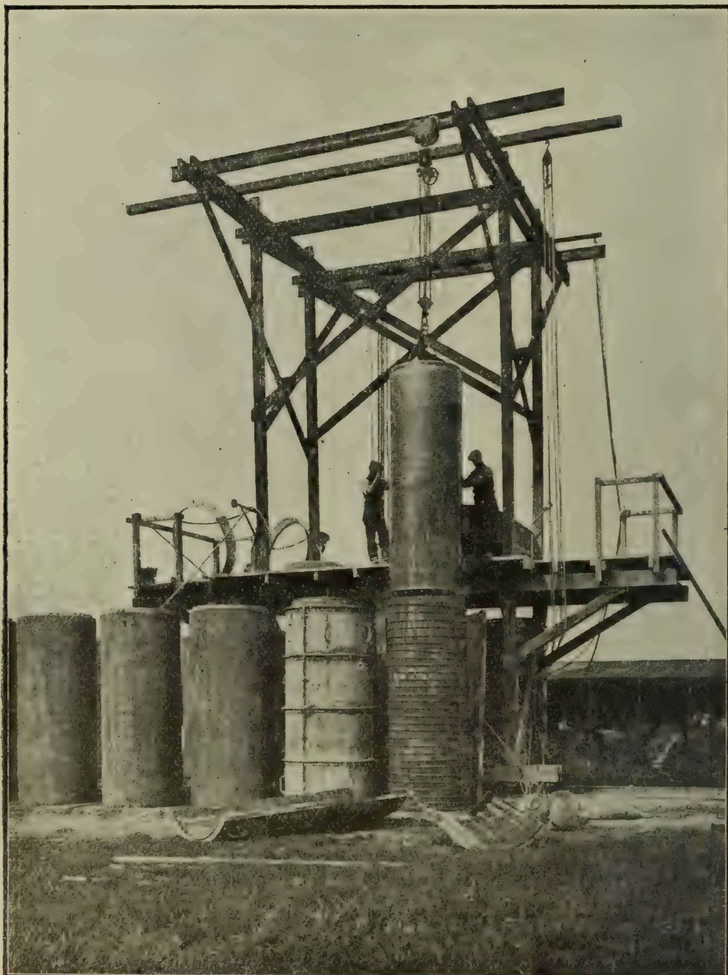


FIG. 27.

if this is used, are first placed vertically on a staging, a collapsible steel core is then lowered and fixed accurately in

position, after which a detachable iron outer mould is placed around the reinforcement and tube.

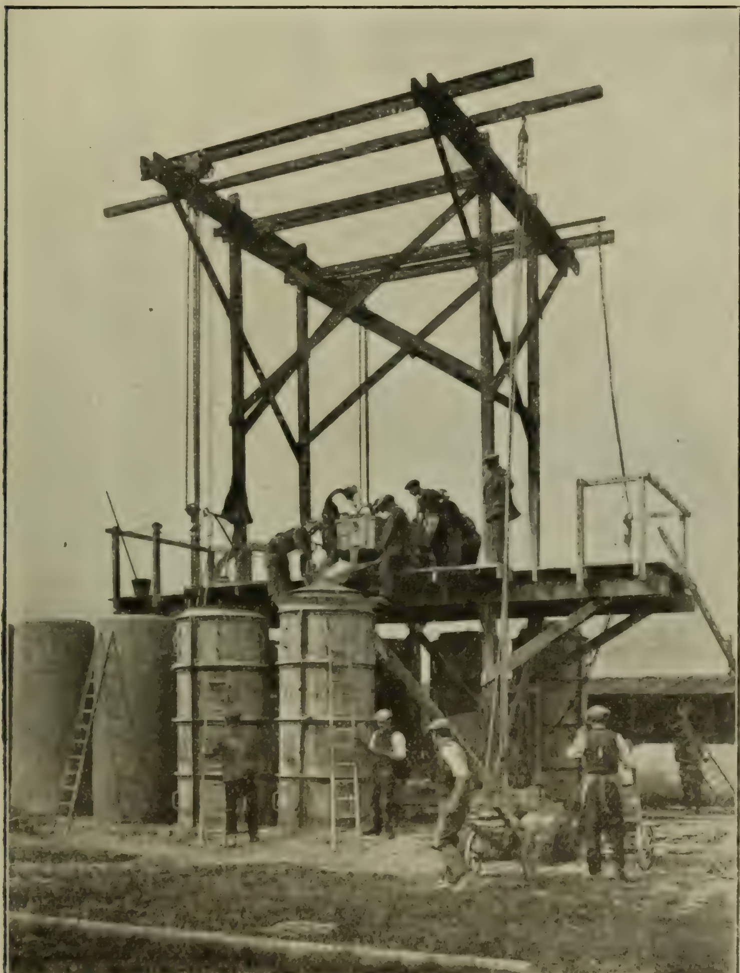


FIG. 28.

The concrete is mixed fairly liquid and is poured into the mould from an elevated staging. While the concrete is being

poured the outer mould is struck repeatedly with a hammer to consolidate the concrete and drive out the air. The process is shown clearly in Figs. 27 to 29, which are taken from photographs of the moulding operations for the 42-in. Bonna pipes for Leeds Waterworks for the conveyance of water from one of the reservoirs to the Westwood Filters under a head of about 50 ft.

Fig. 27 shows the collapsible core being lowered into position for one pipe and the outer mould fixed for another.

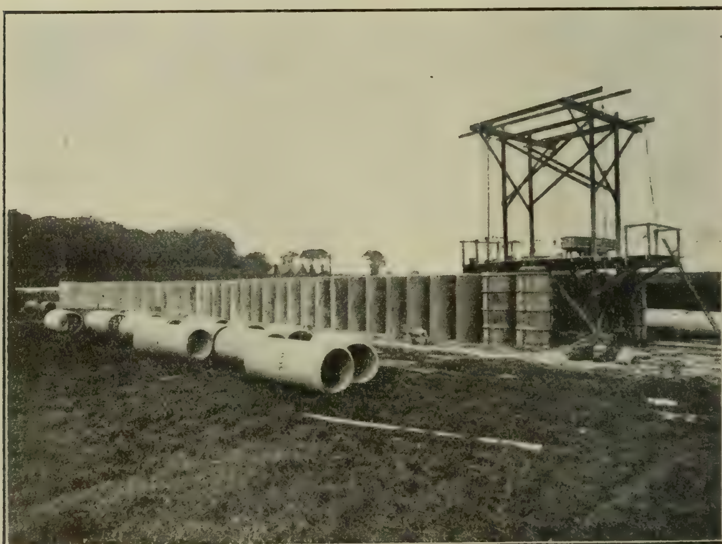


FIG. 29.

Fig. 28 shows the concrete being poured into the mould.

Fig. 29 shows a general view of the pipe field.

Pipes up to about 4 ft. in diameter are moulded in advance and the joints formed with a reinforced concrete collar. For low pressures the collars are simply run with cement mortar, but for high pressures the joint is made with a steel corrugated ring into which are driven lead pipes filled with gasket. The collar is then threaded over the joint and run in the usual manner.

Fig. 30 shows this form of joint.

Large pipes must be formed *in situ*, and in this case great care must be taken to form a good connection when recommencing the work after a stoppage.

When the pipes are moulded in advance it is advisable to use a quick-setting cement in order that the moulds may be removed without loss of time, so that they may be ready for moulding a fresh pipe as soon as possible.

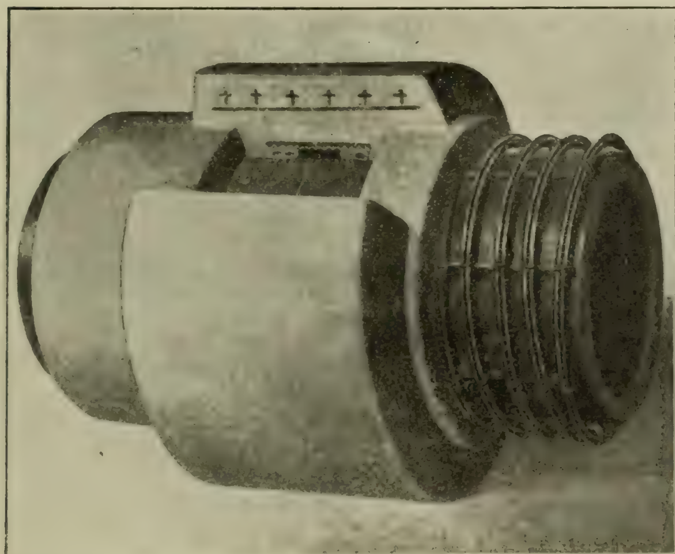


FIG. 30.

After the moulds have been removed the pipes are left standing for about 5 or 6 days, and are then lowered and left lying horizontally until removed for laying.

The pipes should be kept damp for at least one month after the removal of the moulds.

In designing pipes the stress in the steel should be limited to 12,000 lbs. per sq. in., and it is advisable to form the reinforcement with outer and inner spirals when the pipes are of large diameter for the same reasons as given previously with respect to elevated circular tanks.

The thickness of concrete for pipes may be approximately 1 in. per foot of diameter with a minimum of $1\frac{1}{2}$ in.

SIXTY-SEVENTH ORDINARY GENERAL MEETING

WEDNESDAY, MARCH 15, 1916

THE SIXTY-SEVENTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held at the Lecture Hall at Denison House, 296 Vauxhall Bridge Road, Westminster, S.W., on Wednesday, March 15, 1916, at 5.30 p.m.

PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc. (the President), in the Chair.

MR. W. G. PERKINS, District Surveyor for Holborn, then read the following paper :—

SOME EXAMPLES OF DANGEROUS STRUCTURES

The following remarks are not intended as a complete treatment of this subject, but are simply a series of reminiscences descriptive of structures with which I have had either to deal or of which I have had information from those who have dealt with them.

I must not, of course, specify any particular buildings or structures, or I should lay myself open to a series of actions at law.

My own experience, like that of most people, is limited to a certain field, but I intend to mention several kinds of structure with a view to eliciting a discussion which will persuade many to state their experiences, supplemented by communications to our Secretary, giving detailed particulars for inclusion in the TRANSACTIONS of this Institute.

Important failures, such as those of the Quebec Bridge, the Empress Dock at Southampton, Charing Cross Station roof, etc., have been described and analysed in papers read before this Institute by Mr. O. Faber and Mr. F. E. Wentworth-Sheilds, whilst other particular cases have been described in the engineering journals or in papers read before the Institution of Civil Engineers and elsewhere.

Amongst the causes of a dangerous structure are the following :—

(*a*) Faulty construction.
(*b*) Faulty materials.
(*c*) Faulty design.
(*d*) Decay and fatigue.
(*e*) Overloading.
(*f*) Removal of extraneous support.
(*g*) Wind pressure, shock, etc.
(*h*) Fire and explosion, bombs.

I propose to enumerate a number of cases where failures have resulted from one or more of these causes ; generally speaking, several occur at the same time, particularly (*a*), (*b*) and (*c*).

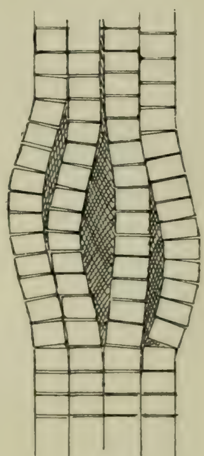


FIG. 1.

Perhaps few of us have had any experience with failures arising from faulty design and materials in modern structures, as a sufficient period has not elapsed for time to have done its work, but there are numerous instances in buildings erected by former generations.

We moderns are often called to account for the supposed inferior manner in which we build, being told that our buildings are not solid like those erected in "the good old times," that we have lost the art of making good mortar, etc. People who make such remarks have, I fear, only an acquaintance with the jerry-builder of the very bad type. The majority of buildings erected in London one hundred to two hundred years ago were constructed in a most inferior manner. The mortar appears to have been compounded with a fat lime, dry slaked, and, judging from the nodules of loam it contains, mixed with a good deal of the "top spit" of the field. Naturally such stuff has, and had, no binding qualities, and to this day is only so much dusty rubbish.

The bricks were badly shaped and easily broken, so much so that in taking down old walls one finds

course after course of what appeared to be headers to be only "bats." This resulted in the walls being built in a series of "half brick" or $4\frac{1}{2}$-in. skins, which sooner or later separated and bulged, as indicated in Figs. 1 and 2, and many a dangerous structure is due to this cause alone. Fig. 3 is an example of the

FIG. 2.

type of brickwork referred to. If carefully examined it will be seen that many of the bricks which at first sight appear to be "headers" are too wide and really consist of portions of stretchers—i.e. "bats."

Fig. 4 is a photograph of part of the basement wall of a five-story building. The four upper stories of two buildings, including their floors and roofs, were supported (?) by this wall, which, as well as bulging,

was out of the perpendicular some 9 in. at its east end and 11 in. at its west end. The piece of solid brickwork shown in the photograph is some new work put in at a point where the load from the rear external wall of the adjacent premises was concentrated by a girder.

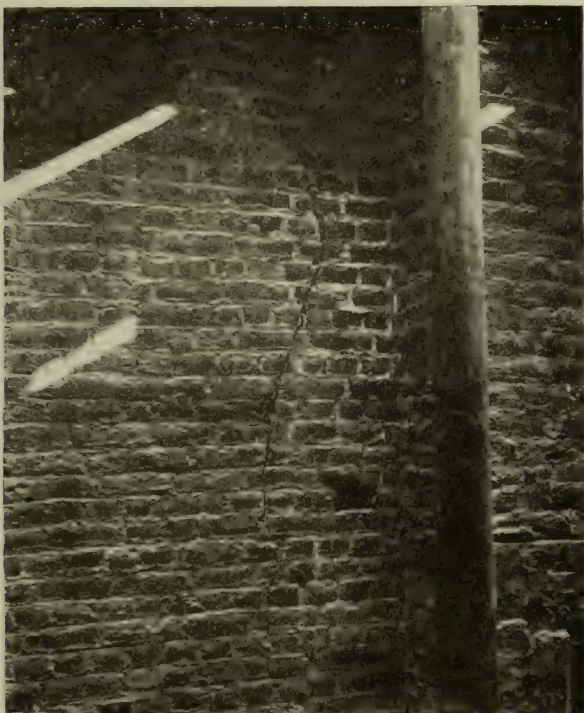


FIG. 3.

Generally speaking, this bulging occurs in the basement story immediately below the level where the thickness of the wall diminishes by means of a "set off" from one thickness to another, say 1 ft. $10\frac{1}{2}$ in. to 18 in. The phenomenon of failure at a sudden change in thickness is referred to by Professor Ira O.

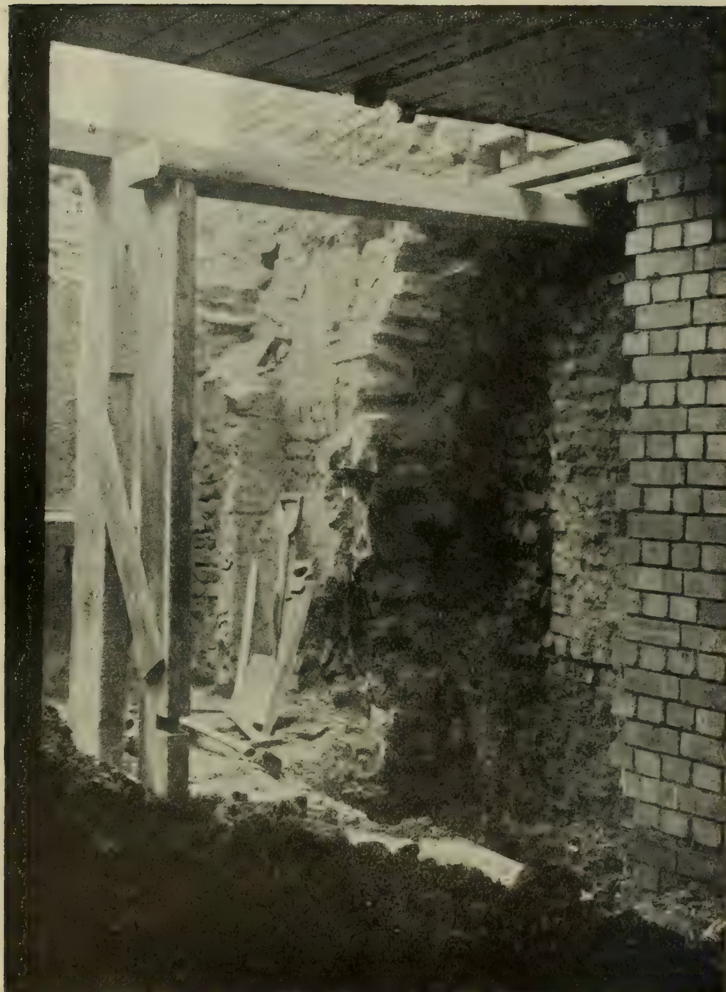


FIG. 4.

Baker in his work on "Masonry Construction," p. 314, 10th edition.

Sometimes large portions of brickwork roughly triangular in elevation and wedge-shaped in section, as shown in Fig. 5, burst from the walls, and when this is the case no time should be lost in needling the walls well above the point where the failure shows itself.

In many old buildings the main beams of the floors

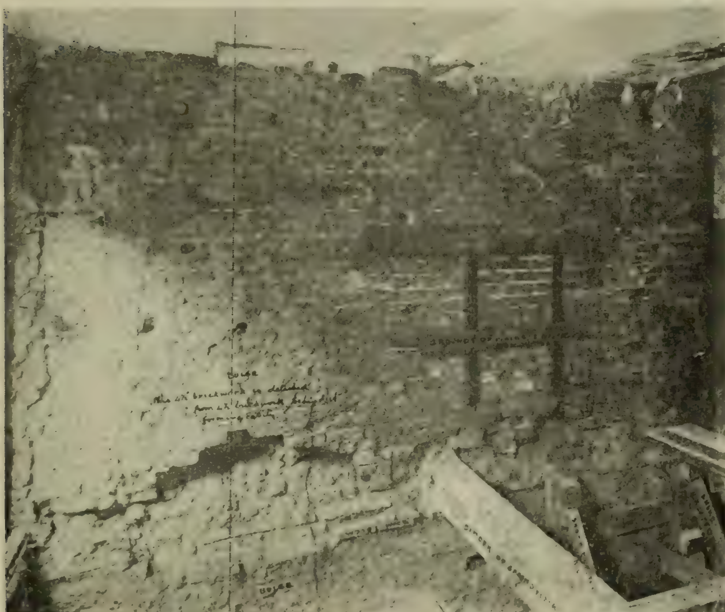


FIG. 5.

are placed diagonally and the loads from the roof and four stories are imposed upon a pier of brickwork about 14 in. square, as in Fig. 6. Needless to say, such piers have crushed.

Several cases I have dealt with have been party walls built with the bricks and mortar before described, about 50 ft. in height from the basement floor line to half-way up the gable. They have failed by crushing in the basement, although the load con-

sisted of only their own weight and that of the dwelling-room floors and furniture on them. The floors have been divided into bays by beams spaced 10 ft. or 11 ft. apart. With 40 ft. in height of 14-in. work and 10 ft. of 18-in. work, the weight per square

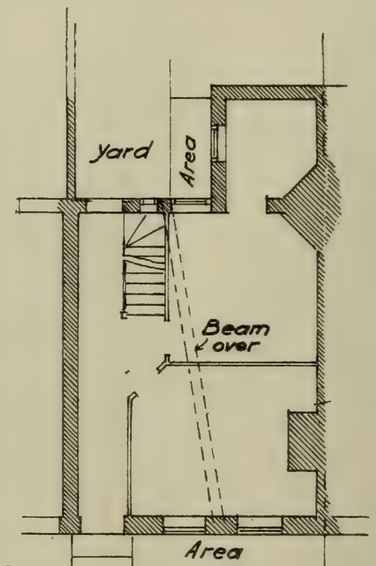


FIG. 6.

foot at basement floor level from the wall itself would be per foot-run—

$$\begin{aligned} 40 \text{ ft.} \times 1 \text{ ft. } 2 \text{ in.} &= 46\tfrac{2}{3} \text{ cub. ft.} \\ 10 \text{ ft.} \times 1 \text{ ft. } 6 \text{ in.} &= 15 \quad \text{,,} \\ \hline & 61\tfrac{2}{3} \quad \text{,,} \end{aligned}$$

which at 112 lbs. per cub. ft.=6,867 lbs. The weight of the joist and boarded floors would be 12 to 15 lbs. per square foot, so that we should have per foot-run of wall 4×10 ft.×1 ft.×15 lbs.=600 lbs.

The superimposed load on the floors cannot be stated with anything like accuracy, as it varies with

the affluence of the tenant (I am speaking of dwelling-houses), but let us suppose it is as much as 50 lbs. per square foot. As the roofs were very steep and covered with thick tiles which absorbed a large amount of water and which were pointed and bedded in mortar

FIG. 7.

in many cases, let us assume they also weighed 56 lbs. per square foot of covered area.

This will give us per foot-run of wall 4 floors and 1 roof $= 5 \times 10$ ft. $\times 1$ ft. $\times 56$ lbs. $= 2{,}800$ lbs. Total load per foot-run $6{,}867 + 600 + 2{,}800 = 10{,}267$ lbs.

The lower part of the wall being 18 in. thick this gives us a pressure of $\frac{10267}{1 \cdot 5}$ = say, 6,845 lbs., or a
If 56 lbs. per square foot superload on the floors is
trifle over 3 tons per square foot as the ultimate load.
too high, then the walls have failed under a pressure of less than 3 tons per square foot. It must be remembered, however, that these walls have been standing

FIG. 8.

some two hundred years, and therefore the failure may be due partly to fatigue.

Strangely enough, this kind of brickwork is always stronger in damp positions, the moisture having enabled the lime to a certain extent to set. Workmen refer to this state as being "waterbound."

Again, I have found a wall of this period built with two skins of brickwork, $4\frac{1}{2}$ in. thick, "tied" together

with wooden laths at intervals of about 18 in. in height, the space between being filled with small pieces of brick and stone. The photographs in Figs. 7, 8, and 9 illustrate a case where the more solid upper portion of a wall wedged itself down into the defective work in the lower stories. Notice that the raking shores were left standing and that the brickwork was of too poor a quality to stand upon the needles.

Fig. 9.

A form of construction frequently found is a shell of $4\frac{1}{2}$-in. brickwork around a stout timber post. You see the brickwork bulging and wonder why a stout-looking pier, perhaps $2\frac{1}{2}$ bricks square and 8 ft. in height, should be failing under a comparatively light load. The explanation is that the timber post has decayed at its lower end and the thin shell of brickwork is unable to carry the superstructure.

A very interesting case of this kind was a building

near Queen Square, Holborn. Having occasion to inspect the premises, I found the walls generally were of the kind already described, and that the piers of the front external wall were yielding. The piers were cut into and, as I suspected, timber posts rotten at the ends were disclosed. These posts extended up to the second floor, and were quite sound in the upper parts.

Sir Francis Fox was consulted by the owners, and it was decided to grout the whole of the walls with Portland cement. Numerous small holes were cut in the interior faces just large enough to insert the nozzle of the grouting-machine. After well soaking and washing the interior of the wall with clean water under pressure from the machine, Portland cement grout was pumped in, and as it appeared at the joints and cracks it was dammed with clay stuck upon the faces of the walls by workmen watching for its appearance.

The whole of the walls were treated in this way with success. Prior to the grouting they sounded hollow, but they now sound and have every appearance of being solid. After the grouting the wooden posts were cut out of the piers and substituted by brickwork in cement.

Some alterations, involving the taking down of a substantial-looking chimney, were taking place in my district.

Above the roof of the building the chimney was rectangular, below, it was triangular on plan, as shown by Fig. 10. When the rectangular portion had been taken down, the part between the ceiling and the floor of the topmost story collapsed and fell on to the roof of a two-story building adjoining, crushing in the roof.

I saw it directly afterwards, and as with the exception of the facing bricks there was only a heap of rubbish (small pieces of bricks and mortar), I came to the conclusion that the large triangular portion had been built hollow—just a shell of 4-in. brickwork, filled up with brick rubbish.

The vibration caused by the alterations, which included a good deal of knocking and the hoisting of girders, had evidently caused this rubbish to settle down and exercise an outward pressure on the 4-in. shell of brickwork, and as soon as sufficient weight

had been removed by the demolition of the upper part of the chimney the pressure burst out the sides of the shell.

Modern brickwork is not without its faults, due principally to the lax manner in which it is supervised and an imperfect knowledge on the part of the specification writer. He will, for instance, require that four courses shall not exceed 12 in. in height, and then select bricks that are very little less than 3 in. in thickness. The consequence is that the bricklayer, to keep to the specification, will put an insufficient bed of mortar in laying the bricks. I have had as much as 3 ft. of glazed brickwork at a time taken

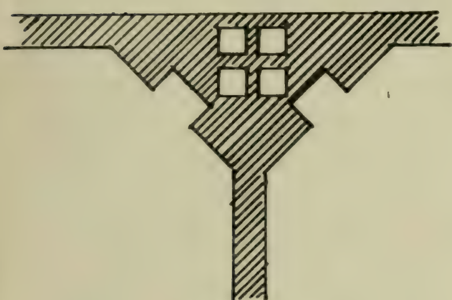


FIG. 10.

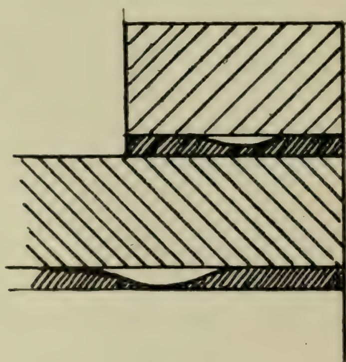


FIG. 11.

down because there was absolutely no mortar in any of the beds. And this was on a first-class job, under architects and a clerk of works. Again, when the bricks have a frog, that frog should be upwards when the brick is laid, otherwise the brick beds only on the rim and its bearing area is reduced by one-third. When the bricks have two frogs one should be "buttered" or filled in before laying.

Unless strictly supervised the vertical joints are never filled solid. Some mortar is rubbed over the top of the course with the flat of the trowel, which may enter the joints some three-quarters of an inch. Particularly is this the case with Staffordshire blue bricks, which are certainly difficult to make solid work with,

as, using the workman's expression, they "swim all over the place."

A habit of the bricklayer, after having spread the bed, is to work the mortar away from the centre with the point of the trowel, thus— 

so that in section it appears as shown in Fig. 11, the centre of the bricks being unbedded. The mortar does *not* squeeze along and make a solid bed as some people argue. To ensure solid work the beds should be thickly spread and the bricks pressed down into it, frogs up, and the whole course grouted up by heaping mortar on the wall and softening it with water from a "gauge-pot" so that it becomes soft enough to flow into the vertical joints until they are full.

Unless the beds and joints are full you cannot expect to get the full value out of your walls and piers, which, by the way, are always eccentrically loaded. The evil is aggravated when the wall is faced with stone, for the mason generally spreads a bed of putty, 3 or 4 in. wide, along the outer edges of the stone he is laying (which may be 14 or 18 in. on bed, such as I have lately seen), points up the back edge, and after having poured some grout down the vertical joints, considers he has made a solid job! Have the stone taken up again and note the result.

Workmanship of this sort leads to "flushing" and the fall of pieces of stone. The bed of every course of masonry should be screeded.

Many people object to slate damp-proof courses, as they become inefficient owing to the slates cracking. They only crack or crush when they have been imperfectly bedded. The usual way is to lay them on a bed of cement (perhaps containing a number of small stones) and *tap* them down with the trowel. This always results in the centre of the slate being unbedded. Walk along such a damp course on your heels, and you will demonstrate to the builder in a striking manner the hollowness of his work.

Use cement made with a sand that contains no small stones, *rub* each slate well into its bed, and you will get a damp-*proof*—not a damp—course.

I am not partial to asphalt damp-proof courses

unless they are of genuine asphalt. Artificial asphalts are brittle when cold but become soft in the heat of summer or near a heating chamber, and squeeze out. This allows settlement, and in the case of a retaining or other wall with a lateral thrust to sustain, may allow sliding on the bed, a result which happened not very long ago "somewhere in the Midlands."

Failures have occurred by using unsuitable materials in the composition of concrete.

There was the case of the reservoir where blast furnace slag was used as the aggregate for its concrete walls. The water affected the concrete so that it became quite soft and rotten, and the walls had to be rebuilt.

Fletton bricks, as our Past President, Mr. Wells, has pointed out, do not make reliable concrete owing to the large amount of calcium sulphate they contain. When this sulphate is in the form of small nodules walls built with such bricks will soon become dangerous. One well-known architect had to deal with a church whose walls became rotten and bulged in consequence. At Dalston, Clapton, and in the Kingsway I have condemned such bricks, and they crumbled away before they could be removed.

Concrete made with breeze is a dangerous material. It is one of the materials specified in the London Building Acts as a fire-resisting material, and as a District Surveyor I have, much against my better knowledge, to pass it. This stuff was used for the floors of sculleries in the first stories of a row of houses built under my official supervision. In every house this stuff expanded, pushing out the walls, some of which had to be rebuilt.

In another district I had to inspect a block of residential flats, the walls of which were bulged at every floor level owing to the expansion of the breeze concrete.

If you use broken brick for concrete see that it is free from any old plastering or Keene's cement, Parian cement, Sirapite, or similar material. Provided this aggregate be quite clean it is one of the best for floor work.

Fire escape balconies should not be constructed

with concrete made of coke breeze. Being exposed to the weather they are peculiarly liable to be affected, and would fail perhaps just at the moment they had to be used.

Our Secretary has told me of a concrete floor which became dangerous in this way: After having set it was covered with one of the jointless floorings now so frequently laid. The acid used in laying this flooring began the corrosion of the steel joists, and the hygroscopic nature of the composition kept the steelwork constantly moist, so that the rusting continued until the top flanges were half rusted away.

Many overhanging cornices of stone or brickwork and cement have become dangerous through being insufficiently weighted or tied down.

The weight of the overhanging portion and its overturning moment must be carefully estimated, and a sufficiently heavy blocking course added so as to obtain a moment of stability of at least 2.

The top of the wall upon which the cornice rests should at the same time be well tied in, for the cornice, though stable in itself, will by reason of the eccentric load it imposes upon the wall tend to draw it outwards. I have had a very recent experience of this in New Oxford Street.

With some stones the soffits of overhanging portions, particularly modillions, brackets, and other ornaments, quickly decay and fall, especially when in the neighbourhood any process involving the discharge of acid fumes into the air is carried on.

From cement cornices the modillions and sheets of cement will fall, and I have noticed this to occur generally after a period of prolonged wet weather.

In my district there are many parapets with balustrades of Roman cement containing iron cores which have rusted, bursting the balustrades so that pieces fall on to the footway.

The type of cornice and parapet most prone to fall is that occurring in a terrace of houses having V-shaped roofs. There is a considerable area of thin wall exposed to the weather on both sides; wet and frost soon destroy the cementing properties of fat lime mortar, and then the overhanging weight of the cornice, plus perhaps a little wind pressure or

vibration, brings the whole lot down, as shown in Fig. 12.

A case that frequently occurs is that of the wall supported upon a beam of timber, which in turn is often carried by timber story posts. In course of time these timbers become decayed, nearly always at the ends, and generally through water gaining access.

Fig. 13 is a sketch of a case in High Holborn. Here a firm of shop-fitters were about to fix a new shop

FIG. 12.

front. Upon inspecting the premises whilst the old front was being taken out, I found that both ends of the wooden bressummer were absolutely rotten for a length of 2 to 3 ft.

The rainwater pipes which came down the face of the buildings had a "swan neck" just above the level of the shop front, the pipes from thence downwards being recessed into the face of the piers separating the premises. The pipes had become partially choked at the swan necks and had leaked at the joints, this having gone on for years. The timber in consequence

became rotten, and the front wall had settled considerably.

It would have collapsed had it not been for a small 3-in. cast-iron column in the centre of the shop front, and the friction against the walls of adjoining premises.

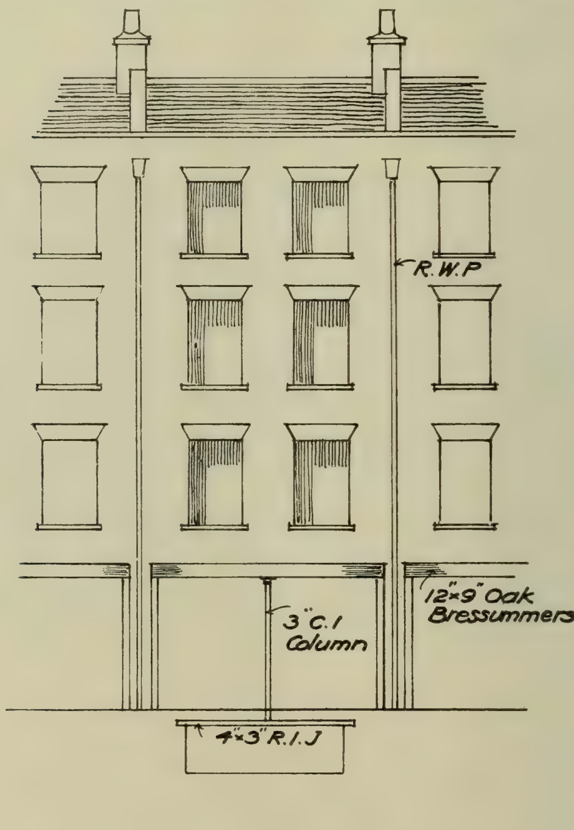


FIG. 13.

The only support to this column was the 4×3 in. rolled iron joist indicated in the sketch.

The condition of the front bressummer caused me to inspect that which carried the main back wall. This I found to be in a still worse state, as the wet

had been getting behind the flashings of the flat roof over the rear portion of the ground story.

A choked or defective gutter over the cornice of the shop front is another cause of rotten bressummers.

To deal with these cases the wall must be needled and shored, "punching up" the solid part of the beam before cutting any holes for needles, and then a new bressummer inserted.

It is convenient at this point to refer to the loading of bressummers. Text-books state that when an imperforate wall is built upon a beam the load on that beam may, owing to the bond of the brickwork, be assumed as the weight of the brickwork enclosed by an equilateral triangle which has the clear span for its base.

It is dangerous to make this assumption, for it cannot hold good unless the bond is practically perfect, and all the vertical joints filled solid with cement. Unless you watch every brick the work is not done in this manner, but as I have already described.

And I would ask, if by the bond all the load of brickwork above the equilateral triangle can be transferred direct to the supports, why should not any portion of the load near the supports or any concentrated loads actually over the supports be distributed over and towards the centre of the span?

The method of building a wall brick by brick with soft mortar ensures, I think, that the whole of the masonry over the span is borne by the beam, and this view is borne out by a case with which I have recently dealt.

At some time a comparatively small opening was made in a wall in the ground story, and no lintel or arch inserted to carry the brickwork over. A crack started from the centre of this opening, and after extending upwards for a few feet became bifurcated, the double crack, some 3 in. in width, extending to the top of the building.

In a reinforced concrete structure you would be required by Regulation 18 to consider that a concentrated load was dispersed over the beam at an angle not greater than 45° with the vertical.

Wooden story posts supporting bressummers decay at the ground line, and are a fruitful source of danger-

ous structures, as a rule easily dealt with, but the following case is interesting.

The building stands at the junction of two streets, and the superstructure was carried upon 2×9 in. timber story posts. The ends of these posts had decayed, and, there being no bracing, the building leaned over towards one of the streets as shown in Figs. 14 and 15. The iron "column" at the corner was a piece of pipe, and did no work.

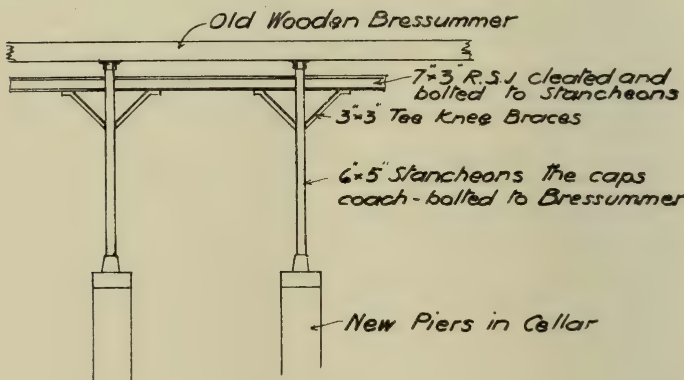


FIG. 14.

I dealt with the case by inserting under the bressummer a frame of steel joist stanchions, tied together near their heads and braced with knee braces, set upon new brick piers and concrete foundations.

You are all familiar with the small suburban house of two or three stories, a cross-section of which is roughly indicated in Fig. 16, and in which one-half of the weight of the roof and floors is imposed upon a central partition of slender timbers lathed and plastered. The timbers of this partition in the lowest story are sometimes found to be in a state of decay owing to the want of a damp-proof course. Rotting at the feet, they allow the floors and roof to sag towards the centre of the building and tend to push out the walls.

By-laws which permit the most heavily loaded portion of the building to be constructed with 2×4 in. or 2×3 in. battens are seriously defective.

Ends of beams and roof trusses solidly built into walls sooner or later decay, and cause dangerous floors and roofs. Ventilation is a *sine qua non* wherever timber is used. A roof boarded, felted,

FIG. 15.

and covered with slate or metal and ceiled will develop dry rot. Such a case, I was informed by the foreman, happened in buildings erected by a certain public authority. Dry rot developed in the ground floor will ascend the timber of stud partitions previously

referred to, and spread all over the first floor, and attack the roof timbers via the woodwork of the first floor partition. Such a case I know occurred at Sidcup. Other cases of floors attacked with this destructive fungus have been reported in the legal and technical papers, where the architects concerned have had to pay damages.

The term "foundation" is sometimes used in a very indefinite manner, and may mean the actual base

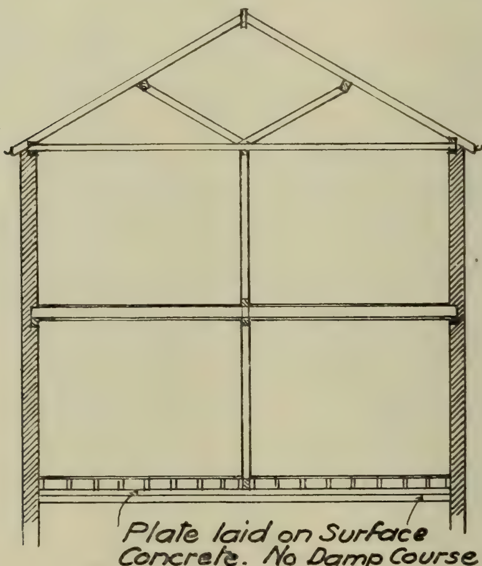


FIG. 16.

upon which the superstructure is reared or the soil itself. When terms such as "bad foundation," "rock foundation" are used, the nature of the soil is evidently referred to; whilst should the term "concrete foundation," "piled foundation," or similar term be used, a form of sub-structure is meant. Why should not the Concrete Institute take the lead, as it has in the case of Standard Notation, and give some of the terms in common use a definite meaning?

It has been suggested that the proper meaning of

the word is the artificial arrangement or construction prepared or made to support the base of a superstructure; whilst the soil beneath should be termed the foundation bed. In this I agree.

The object to be attained is the formation of such a solid base that no movement detrimental to the superstructure can take place after its erection.

This may be done by means of spread footings, rafts, beams, independent piers, piles, cylinders and caissons. With the exception of rock, it is impossible entirely to prevent settlement in the foundation bed, but the engineer or architect may to a very large extent ensure that such settlement shall be uniform, so that cracks and other defects will not appear to a serious extent after the structure is finished.

For the data necessary in calculating the proportions of any of these types of foundation I must, of course, refer you to the text-books, the subject being far too large and beyond my ability to deal with in this paper. Foundations are important, as when defective or insufficient they will lead sooner or later to a dangerous structure.

If a structure has become dangerous owing to insufficient foundation or a bad foundation bed, we must, before proceeding to apply any remedy, ascertain the nature and bearing capacity of the soil upon which we may have to place our load. Methods of ascertaining this capacity are given in Professor Ira O. Baker's "Masonry Construction" and other text-books, and need not be repeated by me. But if you are dealing with a clay soil, I can assure you that it is useless to construct your foundation at any depth less than 8 ft. below the surface of the ground, for the drought of the first fine summer will affect the soil to this depth. You may think this an exaggeration, but I speak from the experience of a good many cases, and I know that I do not stand alone in this opinion.

If the walls of your structure are upright, in long lengths, and you intend to use an ordinary spread foundation of concrete, or perhaps concrete and brick footings, to comply with a by-law, you may undertake the work in short lengths without any shoring, but take care that the soil you have to excavate through

will not escape laterally, as it may if wet, or in the nature of running sand. In some underpinning recently done in this way the sides of the excavation in the London clay would not hold up half an hour, but fell out in cone-shaped masses before the poling boards

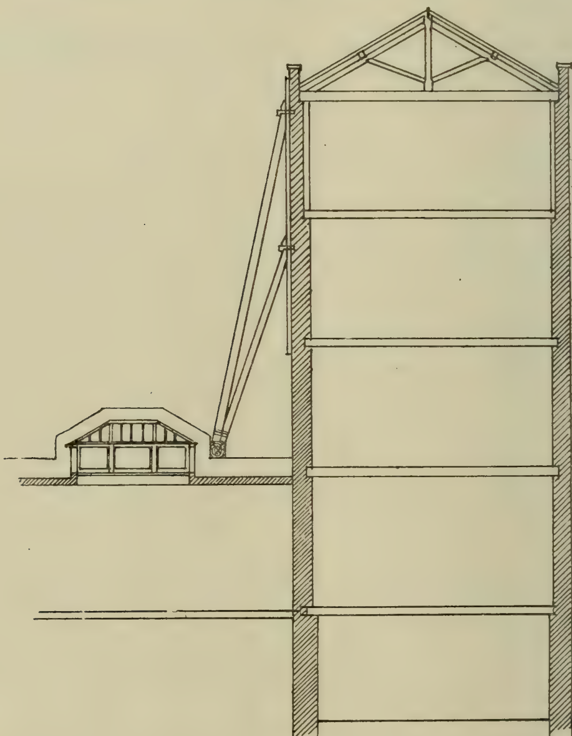


FIG. 17.

could be put in. In such cases it is always advisable to shore with raking shores.

If your structure is supported by piers or columns the superstructure must be carried for the time being upon needles and dead shores, and prevented from getting out of upright during operations by raking shores. Flying shores are also useful when an abutment not too far off can be obtained.

There are cases when it is difficult to erect shores, as, for instance, when you have a high building in a dangerous condition abutting against a lower one belonging to a different owner. He will not, of course, allow you to cut through his roof or to impose any load upon it, but you may persuade him to allow you to place one or more balk timbers across from wall to wall and shore from those, as indicated in Fig. 17. Each case will call for its own special solution.

In dealing with structures rendered dangerous by settlement owing to the insufficient bearing capacity of the soil, I prefer, wherever there is an underlying stratum of firm material, to sink down to it, even though it should be at a considerable depth. The sinking should be in the form of pits (the area of which would be determined by the loading and the bearing capacity of the firm material), filled up with good mass concrete. Then from pier to pier either construct beams of reinforced concrete or fix rolled steel joists, encased in a rich and well-graded concrete, or turn arches.

I am not an advocate of raft foundations, for unless your structure can be symmetrically disposed about the raft or its centre of gravity be made to coincide therewith—a very difficult thing to do except in a symmetrical building—the pressure on the soil will be unequal, the raft will tilt and throw your structure out of the perpendicular. The same thing will happen if the soft soil is not of even consistency and bearing value all over. I have in mind some buildings in the south-east of London where this happened. It can be avoided by the use of reinforced concrete piles. I would not use wood, for this will decay if the soil is not waterlogged or if the water level is altered, and it is almost sure to be by some building or engineering operations in the neighbourhood, or, perhaps, by the sinking of a well. Some time ago our Past President, Mr. Wells, referred to a case where the furnace chimney shaft of a works was rendered dangerous owing to the heads of timber piles becoming charred, the great heat having penetrated through several feet of mass concrete foundations, both downward and laterally.

This reminds me, too, of a case of settlement which once came under my notice, the constant heat of a baker's oven having dried and shrunk an otherwise good clay foundation and allowed settlement of the columns carrying the superstructure.

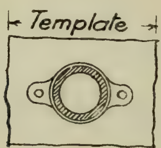


FIG. 18.

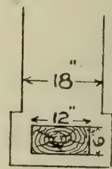


FIG. 19.

In the case of a newly built warehouse, when the first tenant commenced to move in, the floors and roof in the centre of the building sank to an extent of

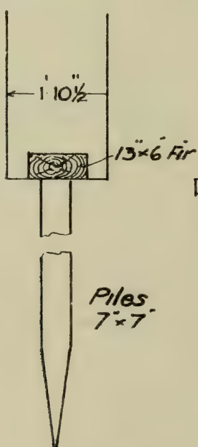


FIG. 20.

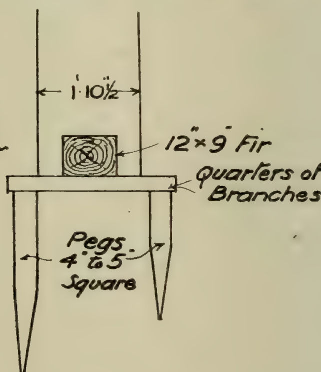


FIG. 21.

something like 7 in., the steel joists deflected, crushed the stone templates, and pushed the walls out of plumb.

It was found that the internal columns had no bases, in the ordinary sense of the word, but lugs similar to Fig. 18, the templates being pieces of old flagstones.

These lugs had broken off and the shafts of the columns had crushed through these templates into the cinder concrete "foundations." It cost a large sum to make the premises secure by the insertion of steel joists and stanchions, with foundations of good concrete carried down to solid ground.

Figs. 19, 20, and 21 illustrate several types of foundations I have met with in my district, where years ago houses were built on land which had been raised in level by the deposit of refuse of all descrip-

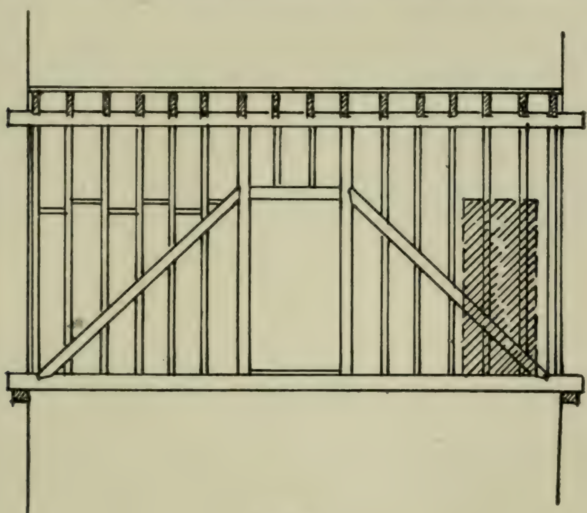


FIG. 22.

tions, for in digging to obtain a foundation one finds, amongst other things, the bones of cattle, horses, and pigs. Excavations made to obtain earth for brick-burning and ponds were filled up in the same manner. The rotting of the wood shown in the illustrations has caused the walls to settle, become out of plumb and crack, and the walls have to be underpinned down to the solid ground. Bond timbers in the walls decay, especially in basement stories and the walls of vaults, and leave the brickwork unsupported. The weight of the floors and roof, always eccentrically applied, then causes the wall to heel over and bulge.

There are other methods of forming timber foundations, shown in the text-books; but I regard them as bad when we can use reinforced concrete, and my advice is to avoid wood for constructional purposes as far as possible.

The continual wet weather last winter saturated many a wall facing south, south-east, and south-west, and at least in one case was the cause of dry rot in the

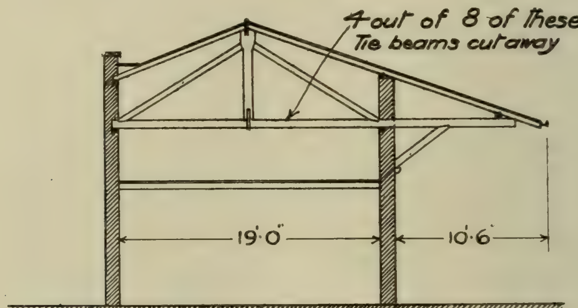


FIG. 23.

skirtings around the first floor, which commenced to attack the joists before it was noticed. The peculiar wrinkling in the skirting-board was quite sufficient to tell one what was happening.

Most of the balconies seen at the first floor level of the older houses in the west central district of

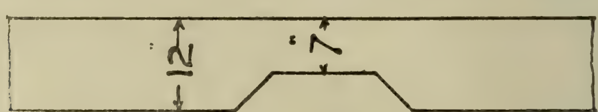


FIG. 24.

London consist of a 3-in. slab of Portland stone, one edge let $2\frac{1}{2}$ or 3 in. into the front external wall, but depending principally for support upon wooden cantilevers. In a great many instances these cantilevers have decayed, and the stone slab has sunk and cracked. In one case wedding guests crowded on to such a balcony, and it collapsed, throwing them all into the basement area.

FIG. 25.

FIG. 26.

FIG. 27.

FIG. 28.

These cantilevers have been painted over and over again, and frequently look to the untrained eye perfectly sound, but generally the little wrinkling I have

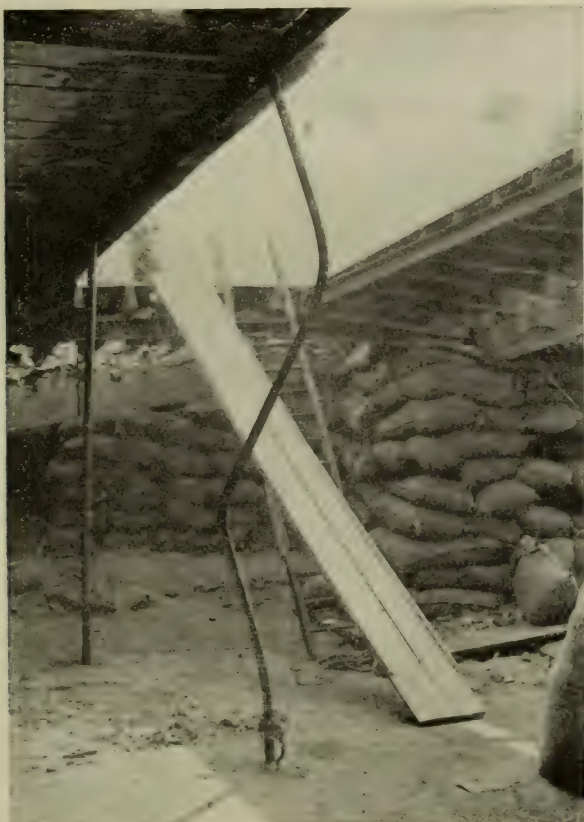


FIG. 29.

alluded to will tell the experienced man that the wood is rotten.

Removing an important part of a structure will speedily render it dangerous. A case in point is the cutting away of the inclined member of a framed partition to form a doorway, as shown by Fig. 22.

This is an alteration occasionally made when replanning old-fashioned houses.

Fig. 23 is a section through a two-story building with a roof which overhung on one side 10 ft. 6 in.

FIG. 30

At some time the tie-beams of four out of eight roof trusses were cut away. The whole building was drawn out of perpendicular, the wall next the overhanging portion of the roof becoming 13 in. and the other 8 in. out of plumb.

A bressummer supporting three stories was cut away

to accommodate a revolving shutter. I happened to pass just as the mischief had been done, and a column had to be inserted to make the mutilated beam secure.

Another case, a fir bressummer, also carrying three

FIG. 31.

stories of a building, had been cut away near the centre of its span, as shown in Fig. 24, so that its original section of 12×10 in. had been reduced to 7×10 in. A new steel bressummer was in this case the remedy.

I have lately seen the top flange of a steel joist entirely cut away and chases 2 to 4 in. deep formed in 6-in. reinforced concrete floors at right angles to

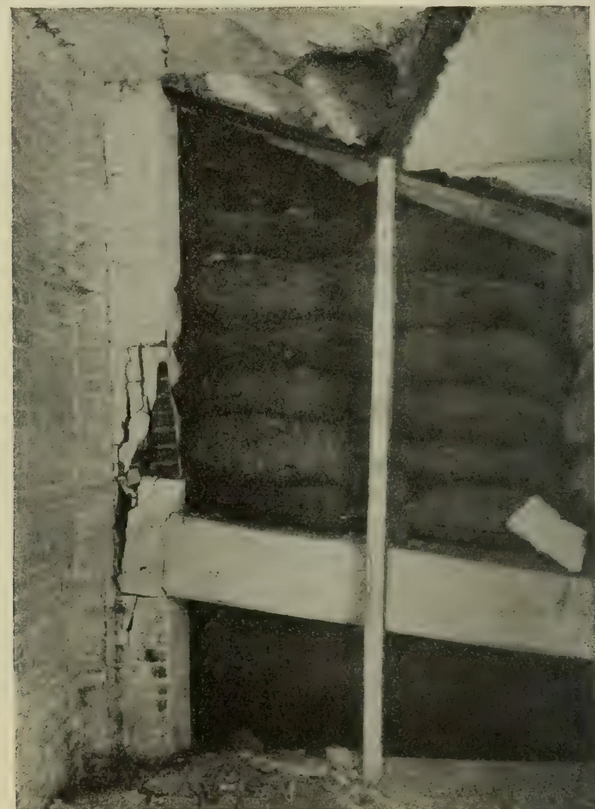


FIG. 32.

the reinforcement for the full breadth of the slabs for the purpose of running electric wires.

Dangerous structures caused by overloading are generally due to the occupier crowding his premises with stock. Figs. 25 to 32 are photographs of a warehouse which had been filled from floor to ceiling in every story with sacks of seed.

A brick pier just below the level of the ground floor which supported the columns carrying the floors and roof failed by crushing, which resulted in the collapse of practically the whole building. You can see from the photographs the extent of the damage done.

No doubt in this case faulty design and faulty construction considerably contributed to the failure, but the pier had done its work for fourteen or fifteen years, and had the occupier consulted a professional man before filling his warehouse the accident doubtless would have been avoided. Fig. 33 is a plan of the pier which crushed.

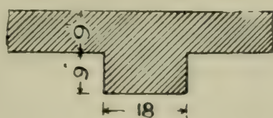


FIG. 33.

The floors and roof of this building were constructed of reinforced concrete, and it was instructive to note that where the concrete was solid and dense the steel was well preserved. It had almost entirely rusted away where the concrete was less dense or contained voids.

Figs. 34 and 35 are photographs of the upper stories in a warehouse, of which the lower floors only were filled with bags of sugar. Before the upper stories were loaded the pier shown in the photographs settled owing to the excessive pressure on the particular soil upon which the building was erected. The manner in which the wooden floors are holding up without the assistance of the metal beams is somewhat remarkable.

A case I had recently to deal with was where a tenant had loaded the first and ground floors with paper—old books and documents. The first floor, constructed with 11×3-in. fir joists and 11×8-in. binders, gave way through the binders breaking. The load was thrown on to the ground floor, which followed suit, and the whole fell into the basement.

It was reported to a District Surveyor that something

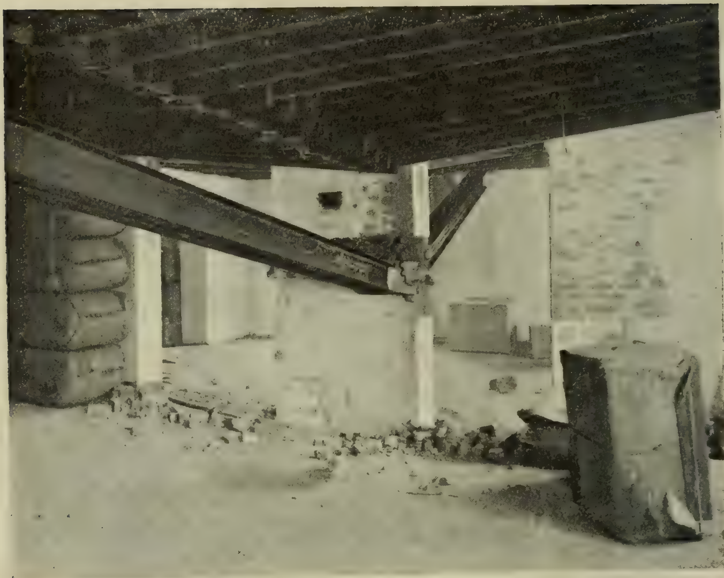


FIG. 34.

FIG. 35.

had given way in a roof, which had a span of 50 ft. Upon getting inside the roof he found that a queen post of one of the trusses had given way, as shown in Fig. 36. It appeared that the trusses were of

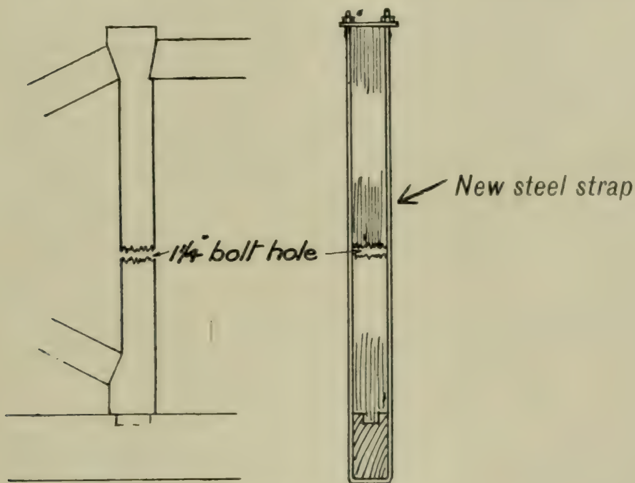


FIG. 36.

oak, being timber from an old ship, and at the point where the queen post had given way was a hole $1\frac{1}{4}$ in. in diameter. Under my colleague's directions, a steel strap, with plate and nuts at the top, was fixed, and the truss has since carried its load without further signs of failure.

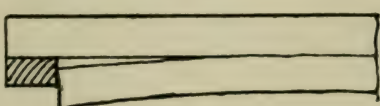


FIG. 37.

In a cabinet-maker's workshop planks were stored in the roof for seasoning, being placed directly upon the tie-beams of the king post trusses. This was all very well until one day, to make room for more stock, an extra quantity was placed in the roof. There-

upon one of the tie-beams broke, and the truss, of which it formed part, collapsed.

Deeply notching the ends of floor joists where they take their bearings will frequently cause them to split, as shown in Fig. 37.

In roofs, too, I often find rafters which have failed in the same way, the split commencing at the ridge and extending downwards for half or three-fourths of their length.

With regard to overloading, there is the well-known case of a football stand, where failure occurred by seventeen of the deal joists supporting the tiers upon which the public stood breaking at the point where they had a bearing over the first of the intermediate

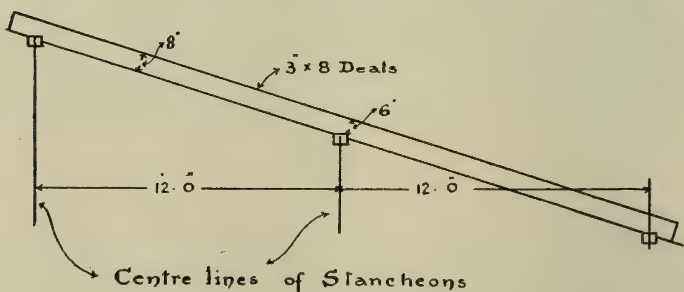


FIG. 38.

supports, counting from the higher end. These joists were spaced 4 ft. from centre to centre and were continuous over one support, thus having two spans each measuring 12 ft. on the horizontal, as shown in Fig. 38. The joists were 3 in. wide by 8 in. deep, and were notched over the central bearing, leaving a working section 3×6 in. This point, as we all know, would be that of maximum bending moment.

Each joist supported ten rows or tiers, which, allowing 16 in. per person, would provide standing room for thirty people. Had each of the persons grouped at this particular spot weighed as much as 11 stone, each joist would have had to support nearly 42 cwt. of crowd. Adding to this 3 cwt. for the actual weight of the joists and flooring, the total load on each joist was 45 cwt. This is equal to

126 lbs. per square foot, without any allowance for movement of the crowd and vibration.

The bending movement was—

$$\frac{45 \text{ cwt.} \times 12 \text{ ft.} \times 12 \text{ in.}}{8} = 810 \text{ cwt.-in.}$$

and we may consider this increased by 50 per cent. to allow for movement—i.e. to be 1,215 cwt.-in.

Assuming the deal used to have an ultimate direct strength of 50 cwt. per square inch, the modulus of rupture by Baker's method would be—

$$50\left(1 + \frac{70}{100}\right) = 85 \text{ cwt.}$$

from which the ultimate resistance of one joist would be—

$$\frac{85 \times 3 \times 6^2}{6} = 1{,}530 \text{ cwt.-in.}$$

It will be seen that the fibres of the joists were stressed almost up to their breaking strength, and from their failure it may be assumed that this was actually the case. It can also be shown that the shearing stress was more than the ultimate shearing strength along the grain.

The records of the London County Council if available would show many cases of failure by overloading the floors in buildings of the warehouse class. Such buildings may have been of sufficient strength for the purpose for which they were originally designed, but in course of time the owners or tenants forget all about this, or they change hands, and there comes a day when it is necessary to house an extra large stock. The stories are packed from floor to ceiling, and then the beams break or the walls bulge and crush, or, as in one case, the corbels by which the ground floor was supported were sheared off and the whole floor fell into the basement.

We are constantly reminded of the insufficiency of old structures to carry modern loads by the notices affixed to the bridges which carry our roads over railways, canals, water supplies, etc. Good

enough no doubt in their day, they are certainly inadequate for the loads specified in this Institute's Report on the Loading of Highway Bridges.*

Digging trenches for drains and sewers without sufficient timbering has been a fruitful source of dangerous structures. Many a wall and building have collapsed from this cause. Cases at Finsbury Park and Wood Green, where in each instance a row of shops partially collapsed, will doubtless be remembered by many of you.

Fig. 39 is a drawing shewing how a builder sought to keep the wall of a building in position whilst he undermined it during the course of some drainage operations, and Fig. 2 is a photograph of the shores after they had commenced to slip.

Extreme care should be taken when working in bad ground, and the scantlings of the timbers should err on the large side, as it is difficult to calculate the pressure to which they will be subjected. If the ground contains water, the pressure may be equal to a fluid weighing 100 lbs. per cubic foot under pressure to the extent of the weight of the adjoining buildings. The struts which fix the walings, from the very manner in which they are fixed by driving in, are eccentrically loaded by the earth thrust, and in my opinion, a system of cross-bracing and vertical ties should be provided to the struts in all deep trenches (of course, at intervals sufficient to leave working room), so that the timbering would not fall to pieces should a movement of earth occur.

In the same category come excavations, such as for the sub-basements and basements of new buildings, which may be 40 ft. deep or more, and here, again, particular attention must be given to the shoring, especially when the street is a narrow one and there are buildings on the opposite side.

Unlike a trench, an excavation for a basement may be too wide to strut with horizontal timbers, and as you dig so you remove the soil supporting the feet of any timbers inserted when the work was first started.

Any considerable movement in the shores allows a subsidence that will cause a fracture of the water main ; there will at once be a washout—electric cables and gas mains will be broken, the electric current

* See *antea* P. 24 et seq.

setting fire to the escaping gas, followed most probably by the collapse of buildings contiguous to the excavation.

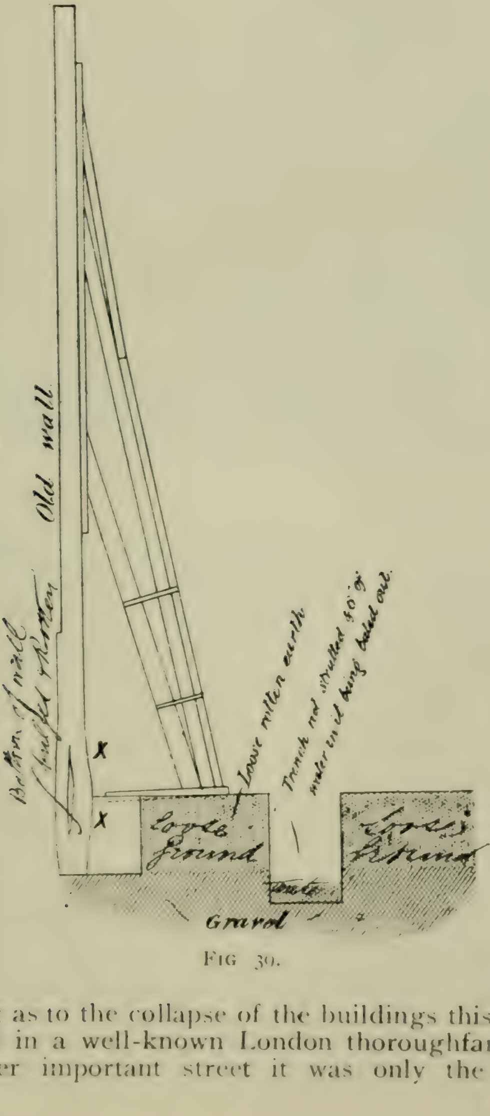


FIG 30.

Except as to the collapse of the buildings this actually happened in a well-known London thoroughfare. And in another important street it was only the prompt

action of the District Surveyor which prevented such a collapse.

In the first case the excavation had reached a depth of about 20 ft. below the street level for a length of 50 ft., and had been strutted with 7×7 in. timbers, some 18 ft. long, spaced 14 ft. centre to centre. The attention of the foreman had been called to the inadequacy of this timbering, but steps were not taken in time to prevent the failure.

In the second case the District Surveyor wrote to the contractors, but, having no jurisdiction, could take no action until buildings were rendered dangerous.

In a third case excavation was being done under the supervision of a superintendent, but again, owing to insufficient consideration having been given to the timbering, a slip occurred ,and a portion of the roadway fell into the hole.

Yet another case was that of a retaining wall, 60 ft. high, which commenced to slide, but was brought up by the timbering, which happened to be very strong. The movement was attributed to the too rapid erection of the wall and the removal of centering before the concrete had properly set.

I have seen many large basements and sub-basements excavated, and in almost every case the surfaces of the adjoining streets have been cracked and the vaults of opposite buildings ruined.

Figs. 40 and 41 are illustrations of the result of the inefficient timbering of basement excavations.

Professor Ira O. Baker states that in Chicago some of the largest buildings have settled owing to the flow of the plastic clay soil towards deep excavations opened across the street.

I suggest that the best way of dealing with such excavations is to dig a trench, timber it in the ordinary way, and brace such timbering ; then build your retaining wall and afterwards excavate the " dumpling." You can do the work in sections if it is impossible to dig the trench for its full length at once. The work for an important building in my district was completed in this way because the surrounding streets showed signs of settlement, owing to the excavations having been commenced in the ordinary manner.

Occasionally a water main will burst, filling vaults

FIG. 40.

FIG. 41.

under the roadway and basements of houses, bulging and fracturing the arches and head walls of the former and the party walls of the latter.

The presence of an old wall or cesspool, planked over and covered with earth, long forgotten and then built upon without troubling as to the foundation bed, has often led to trouble.

After several old houses in one street had been condemned as dangerous structures, the owner of his own accord took down the remainder of the row. When digging trenches for the foundations of the new houses that were afterwards erected on the site, literally dozens of such cesspools were found, two of them sometimes adjoining like the figure 8.

A leaky drain running through made-up soil under the foundations of a building caused the soil to settle, the walls to crack, and the voussoirs of the arches to become loose.

On one occasion I found the sole support of a massive chimney breast projecting 4 ft. into the room at the first floor level to be simply the floor boards and common joists! There were no corbels under the jambs, and the breast was not even bonded to the wall against which it had been built.

The construction of subways and railways in tunnel, tube, or cutting may account for many settlements in buildings adjacent to the line of route by actual movement of the soil or by withdrawal of water from the strata.

A church in London was fractured from its foundations to the ridge of its roof and otherwise very seriously damaged by settlement of the earth at one end of the site, due, in my opinion, to operations of this nature, there being more than one tube railway near by. When the foundations were uncovered the soil, which usually contains a certain amount of water, was found to be quite dry.

An architect friend informed me of the erection of a heavy building on the marshy land near Tilbury, close to a much lighter building, which resulted in an upheaval of the surrounding soil and the cracking of the walls of the latter.

Wind is a frequent source of dangerous structures. Formerly its action seems to have been disregarded by

designers, but the Tay Bridge disaster brought its reality home to the authorities, and it is now always allowed for in bridges and other important structures. Even to-day in buildings enclosed with walls of the thickness specified in local building acts and by-laws no account is taken of wind pressure. In the South of London I have seen a modern building which collapsed during a gale. The cast-iron column which supported one angle of the premises broke into four pieces at the time of the collapse, but this may have been caused by its fall to the ground or the falling débris.

During the gale which occurred on December 14, 1907, one of the legs of a Scotch derrick crane about 117 ft. in height, used in the erection of a large building not far from here, became affected by the wind, and the whole of the structure, together with the platform and crane, the jib of which was about 60 ft. in length, commenced to sway. Before eleven o'clock the crane, according to the *Daily Telegraph*, was swaying 5 to 6 ft. in the wind, and threatened every moment to collapse. Notwithstanding the immense risk, guy ropes were attached by the contractor's workmen to the defective leg, and made fast to the stanchions of the building, which extended down to the sub-basement.

The primary cause of the leg becoming defective was, I believe, the soaking of its foundations by heavy rains.

In 1893 a large crane in the City suddenly swayed and toppled over, falling into the public way, fortunately without any fatal result. I have no information as to why it fell.

Not long ago one of the anchor chains of a Scotch derrick in my district broke owing to the snapping of a faulty link. The crane at that moment was lifting several tons of steel joists, and together with its load fell to the ground, killing one man and injuring others.

These derrick cranes are erected without official supervision by men who, though skilful in the actual building, have no knowledge of stresses. The deals used in their construction have generally some shakes along the centre of their length, and even if sound when first used, the sun and wind soon cause such

shakes to appear. This is particularly likely to affect the cross braces, and if the shakes are in the line of the bolt-holes the members will work loose and the structure become dangerous. The long diagonal braces staying the "king leg" to the back legs should be steel rods with turn-buckles for tightening up, but they are usually half timbers, lapped and bolted near the centre of their length, inefficient owing to their lateral flexibility and the liability of the grain to detrusion.

Another case is that of a football stand which had its roof blown away into the next field, and there are many recorded cases of the removal of roofs by the wind and the demolition of walls, gable ends, chimney stacks, tall chimney shafts, flagpoles, etc.

The London Building Acts Amendment Act, 1909, requires a steel-frame building to be so designed as to resist safely a wind pressure in any horizontal direction of not less than 30 lbs. per square foot on the upper two-thirds of the surface exposed to wind pressure. Even this is objected to by architects of the "old school," who urge that in London buildings are so close together that the wind pressure cannot attain this figure. I have, however, seen a long fence wall, 14 in. thick and 10 to 12 ft. in height, opposite the house in which I was living blown down. Working this out, neglecting the strength of the mortar, you will find that the pressure must have been in the neighbourhood of 15 lbs. to the square foot. And this was not a lofty structure, but one near the ground, with trees and hedges close by on the windward side.

During the gales which occurred in the first week of January this year the roofs of buildings at Tooting and Hanover Square collapsed, and were illustrated in the daily papers, chimneys were blown down, and a vast amount of damage done to buildings throughout the country.

Sky signs in London are now things of the past, but this is what Mr. A. T. Walmsley said when giving evidence for the London County Council before a House of Commons Committee :—

"It would be possible to make a sky sign safe over a new building where piers were built up purposely to secure it, but in the case of nearly all the London sky

signs then existing, the building was not calculated to resist the extra strain nor the lifting action of the wind braces on the eye-bolts driven in to hold them. A guy loosened in mischief, or by decay of inferior brickwork, a perishing bolt, or the giving way of a frail roof, would be sufficient to give such structures a fair chance of doing damage."

During a gale which occurred in November 1893 a factory chimney was blown down at Huddersfield, killing two men. From the evidence given at the inquest it appears that the chimney had been built some nineteen years, and was of brick, square on plan, 6 ft. $1\frac{1}{2}$ in. wide at the base, the walls being 18 in. in thickness, including a firebrick lining, which was built in as part of the structure for a height of 11 ft. The sides, for a height of 21 ft. 6 in., were vertical, but from that point the thickness of the walls was reduced to 14 in., the sides were battered to the top, where the width was 4 ft. 6 in., the total height being 75 ft. About two and a half years before it fell the chimney was raised to 105 ft. in height, the new portion being of 9-in. brickwork, and battered to 3 ft. 6 in. at the top. The original chimney had therefore a height of about twelve times its width at the base, while the raised chimney was about seventeen times the base width, and it had a slight cant in the direction in which it fell. As you are aware, in London the height of a square factory chimney is limited to ten times its width.

The place where the fracture took place was 32 ft. above the ground line, and was abutted to a height varying from 30 to 40 ft. by adjacent buildings on three sides.

The disaster was attributed "partly to the height of the chimney being too great as compared to the width at the base, but principally to the fact that it was abutted against by the walls and roofs of the adjacent buildings on three sides, which gave rigidity to the structure for a height of 30 to 40 ft., whereas such a chimney should be perfectly free to oscillate in all directions when shaken by the wind."

I am indebted to *The Builder* of December 23, 1893, for these particulars, where it was shown that the wind pressure necessary to just overcome the

effect of the weight of the chimney at the point where it failed was 28·87 lbs. per square foot.

As to whether the principal reason given for the failure was entirely right I must leave you to judge. Had the chimney been without the abutments it would certainly have been blown down, whilst had the abutments extended to the top it would not have fallen.

In February 1892 a mill chimney at Cleckheaton, built in 1859 under the superintendence of a joiner as architect, collapsed and killed fourteen persons, besides damaging a considerable amount of property. When the height of the chimney had reached 120 ft. it overhung in a south-easterly direction to the extent of nearly 7 in. Instead of taking down and rebuilding, the "constructors" straightened the shaft (no particulars of the straightening operation are published).

In a report to the coroner Mr. J. Waugh, C.E., of Bradford, said that the bricks with which the chimney had been built were, with the exception of the firebrick, unfit for the purpose of building a chimney even if constructed of solid brickwork. He was also strongly of opinion that from some cause the bricks and the mortar in which they were laid had undergone deterioration owing to moisture, and he attributed the fall to four reasons: (1) bad bricks, (2) weak construction, (3) deterioration by moisture, (4) attempted repairs. Sooner or later, however, it would have fallen, apart altogether from the fourth head.

The total weight of the structure was about 500 tons. The weight of the two shells above the solid was 480 tons. The outer shell alone weighed 325 tons. The distressed area on the easterly and northerly sides was equal to half the circumference. Therefore, if the chimney had been vertical, the weight above half the circumference would equal 162 tons, or 6·33 tons per square foot; but the chimney leaned over on this side, and it was impossible to say how much the load was increased by this fact without a knowledge of the amount of departure from the vertical.

In March 1893 two factory chimneys fell, one at St. Helens and the other at Widnes. The former, which collapsed to the foundation, was 300 ft. high and 24 ft. in diameter at the base and 12 ft. at the top, built in 1850-1. It was said to have been out

of repair, and that the very day before it fell it had been strengthened by an iron band. The Widnes chimney was only 90 ft. high, and the uppermost part fell whilst being repaired.

"Both chimneys appertained to chemical works, and in both apparently the brickwork had been eaten into by the acids and gases emitted from the works."

I referred to *The Builder* for the foregoing particulars, and I have mentioned them, as the cause of collapse was investigated by an engineer.

Runaway motor vehicles have sometimes been the cause of a building becoming dangerous. One I recollect breaking clean in two the cast-iron column supporting the corner of a building which stood at the junction of two streets. Fortunately there was a stanchion about 5 ft. further back from the corner which stood the increased load thrown on to it until another column had been fixed, and the bressummer was also able to do its work as a cantilever.

On another occasion a motor charged and knocked away the raking shore which had been fixed to steady a building after the demolition of its neighbour.

Frequently a wagon with an overhanging load or a skidding motor 'bus will overthrow a gas lamp or an electric light standard, which, of course, is bound to fall on to the public way.

I need hardly say anything with regard to fire—its effects have so often been described and illustrated.

During the raids last year explosive bombs were dropped in certain places, but it is perhaps inexpedient to discuss the result yet awhile, as the authorities would object.

My experience with gas explosions has been limited to minor cases which resulted only in the fall of plaster ceilings, so that I look to some of you for information under this head.

Amongst other things I might just mention the following: Slates and tiles which get loose and rattle down into the streets on a windy day. Although comparatively small, these things can do serious damage to the human being.

Plaster ceilings on laths which weigh 5 to 7 lbs. per square foot. The keys gradually break, and finally a large slab falls. I have seen quite a square yard lying

on the pillow of a bed, which would not have benefited the sleeper had that bed been occupied when it fell. I was once examining an alleged dangerous ceiling when the defective portion, about 12 square feet in area, fell all around me. I then had to certify the structure as not now dangerous!

Pavement lights with cast-iron frames, supported on three sides only, sometimes break on the unsupported edge. They should always have a small joist or other efficient support on the fourth side—it need not be so large as to obstruct the light. Occasionally also a heavy vehicle has run across the footpath on to these lights and broken through.

Flagstaffs become rotten near the base, and where the iron stays are bolted to or clip around them.

Rainwater pipes have a habit of splitting and dropping a piece of themselves, and cast-iron eaves gutters fall through screw-heads rusting off, holes becoming enlarged, or the backs breaking out. Snow sliding off a roof will break gutters in this way.

Cemented reveals and soffits of arches become detached and fall.

Voussoirs of gauged arches become loose when they have not been joggled and grouted with Portland cement—in cheap work they are simply dipped in a box of lime putty.

Terracotta ornaments in a state of incipient tension split off and fall, and so do stone griffins perched on parapets. In a central thoroughfare one of these things fell, narrowly missing a District Surveyor who happened to be passing.

The Secretary has suggested that I should deal with "earthquakes and the like." I have had no experience of earthquakes, but I recollect that in 1893 a landslide occurred at Sandgate. The disturbance was felt along the foreshore for a length of about 2,800 ft., and extended on to high ground for a short distance inland. Between high- and low-water marks there were visible signs of a slight upheaval, a layer of blue clay being obtruded through the line of fracture, with fresh water in some places issuing from the crack. A writer in *The Builder* expressed the opinion that the disaster was merely a landslip, caused primarily by the slipping of the Hythe and Sandgate bed over the

Atherfield clays, the effect of the slip being materially aggravated by the treacherous nature of the Sandgate beds, which immediately the landslip began commenced to break up. The results of the slip, *inter alia*, were that the majority of the buildings in the town were more or less damaged and the gas and water mains disturbed. Houses slipped away from each other, leaving gaps between, whilst the walls of other edifices bulged out and were on the point of falling.

The shoring of dangerous structures is too large a subject to deal with in the limits of this paper, particularly as the practical part is well treated in Mr. C. Haden Stock's "Shoring and Underpinning."

The constructional details of raking and flying shores illustrated in this work are those employed by the London County Council contractors and their predecessors. I do not think they can be improved, and long experience has shown them to be reliable when applied to the type of building that has hitherto constituted our dangerous structure in London. Doubtless it will be necessary to evolve other methods if and when lofty modern steel frame and reinforced concrete buildings become dangerous, especially as good balk timber is becoming more and more scarce.

But the treatment of the theory of stresses in shores given in the work referred to, and by most writers on building construction, is, I submit, wrong in principle.

In "Shoring and Underpinning" the stresses are found thus :—

A horizontal force F is assumed to be acting near the top of a vertical wall, tending to overturn it about its lower edge, its moment being F H, H being the height of F above the base. This is to be resisted by the weight of the wall acting vertically at its centre and having a moment of $\frac{W t}{2}$, W being the weight and t the thickness of the wall.

When these forces balance the wall will be about to fall over and the two moments will be equal, therefore—

$$F H = \frac{W t}{2}$$

In order to restore the wall to its original condition

before the force F acted upon it some way has to be found to balance it, this being done by means of a raking shore, the head of which is at a height h above the base of the wall; then by tightening up the lower end of the shore a horizontal pressure P is to be applied such that its moment about the base balances that of F, or—

$$P h = F H$$

$$= \frac{W l}{2}$$

whence—

$$P = \frac{W l}{2 h}$$

If the shore presses against the wall with this force P, there must be a reaction of the wall equal and opposite to P, so that P represents a horizontal pressure against the head of the shore. In order that the shore may have its full effect in counteracting the outward thrust or reaction P it is essential that it should be prevented from sliding upwards by having a sufficient weight of wall above its head, so that when the pressure P comes upon it it may be kept immovable by the superincumbent load, which we will call L. Also call the weight of the shore w_s. Then the sum of the moments of L and w above the foot of the shore are to balance the moment of P about that point—

$$P h = L d + \frac{w_s d}{2}$$

d being the distance between the base of the wall and the foot of the shore, whence—

$$P \sin \theta = \left(L + \frac{w_s}{2}\right) \cos \theta$$

θ being the angle of inclination of the shore.

From this is obtained—

$$L = P \tan \theta - \frac{w_s}{2}$$

and—

$$P = \frac{2\,L + w_1}{2 \tan \theta}$$

These two forces having been determined, the compression C down the shore is given as—

$$C = L \sin \theta + P \cos \theta$$

(Why not simply L cosec θ, or P sec θ, or $\sqrt{L^2 + P^2}$?)

The writer then determines the magnitude and direction of the resultant R of all these forces, its point of action being shown at the foot of the shore, and he says in Chapter II that its direction is between L and P, that it does not act directly down the shore itself, but is always outside the angle the shore makes with the horizon.

A formula is also given for a bending moment in the shore due to the action of L, P, and w_1. Now I think it is wrong to estimate the horizontal thrust at the head of any shore by reference to the stability of a *vertical* wall to which it is to be applied. A vertical wall will exercise no pressure upon a shore until it begins to rotate about the base or some bed joint, when its moment of stability at once ceases to be $\frac{W\,t}{2}$, the foundation of Stock's theory. Suppose the wall to be out of plumb so that its centre of gravity overhangs the outer edge of the base on the side next the shore, a condition constantly found in practice, the wall not falling owing to the floor joists and other members of the building which have acted as ties.

In such a case W gives no moment of *stability* such as was used by Stock to find P, but one of *overturning*, and the greater the load L at the top of the shore the greater the overturning moment becomes.

If the wall to be shored is not perpendicular, its weight will have an overturning moment equal to W*d*, where W is its weight as before and *d* the distance its

centre of gravity overhangs the centre line at its base, and the stress down the shore will be $\frac{W d}{a}$, where a is the lever arm of the shore about the same point.

This may be written—

$$C_w = \frac{W \sin a}{\sin 180 - (\theta + \phi)}$$

where ϕ is the angle of inclination of the wall.

Whether the wall is vertical or out of upright any inclined force P, such as the thrust of a roof truss or the storage of any material that would exercise a

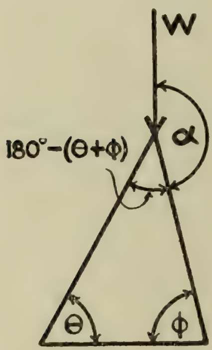


FIG. 42.

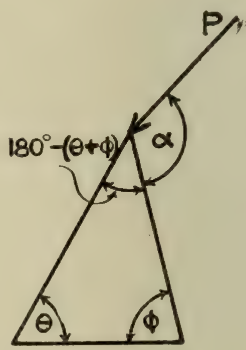


FIG. 43.

pressure against it, or wind pressure, must be calculated and its direction found. The stress in the shore can then be found, thus—

$$C_p = \frac{P \sin a}{\sin 180 - (\theta + \phi)}$$

a being the angle included between the direction of P and the wall, θ the angle of inclination of shore with the horizon, ϕ the angle of inclination of the wall with the horizon. This must be added to the stress due to the weight of the wall.

In all cases, however, it will be simpler to draw a diagram and resolve the forces in the usual way.

I have neglected the effect of the weight of the shore itself, which is small compared to that of the wall, and I have for the same reason taken no account of its negative moment in opposing the overturning moment.

We must, of course, have sufficient weight above the head of the shore to prevent it rising, or we can continue the wall plate downwards and insert a needle through it into the lower part of the wall, the area of any needle being made sufficient to withstand the shear, which is equal to the algebraical sum of C_w and C_p resolved parallel to the wall, less the friction of the wall plate, which, however, would not be taken into account in practice.

Should there be any floor beams or girders imposing loads on the wall, the tendency of the wall to turn outwards will be decreased so long as the line of action of the loads falls on the inner side of the centre line of the base, but when that line of action falls on the outer side it adds to the overturning moment, in which case the floors and beams should be supported by a system of inside vertical shores to relieve the wall of the load, leaving to the raker the wall only.

When the raking shore comprises several members the thrust upon each will be approximately proportional to its lever arm a, so that the sum of their moments is equal to the moment of the wall. (I say approximately because a continuous wall plate will somewhat vary the loads borne by each member.) The lowest member will therefore be the most heavily stressed.

I need hardly caution you against tightening up a shore to such an extent that you thrust the wall or building over in the opposite direction! Care must be exercised in this respect when the floor joists run parallel to the wall or when there is a staircase or well of any description on the inner side, so that there is a considerable height unsupported laterally.

At the risk of somewhat labouring the subject, I must call attention to another curious argument in Stock's treatise.

In Chapter IV, on horizontal or flying shores, an illustration is given of two similar buildings, one of which has to be secured by such a shore extending across the space which separates them, and, quoting

from the letter-press, " of the two houses represented, the one on the left hand is secure and needs no shoring

FIG. 44.

at all, having been built independently of the house that has been cleared away, or, in other words, the return wall belongs to it exclusively, and has not been

shared as a party wall by the house adjoining, consequently the flying shore has to resist the thrusts of the opposite house only. But when both are party walls it will be best to allow sufficient strength in the shore to resist the thrust of *both houses together* ! " In other words, you add the thrust and the reaction together to arrive at the stress in the shore, which is absurd.

When the base or lower part of the wall is in the condition of one of the walls described in the early part of this paper, dead shores and horizontal needles inserted at a sound part of it must be used. Rolled steel joists are frequently used for needles in such cases.

With regard to the scantlings of shores, you must determine the sections by the usual beam and pillar formulæ after having calculated the loads and stresses.

Shores themselves often become dangerous owing to the timbers shrinking so that the head of the raker gets free of the needle, when it may fall over sideways.

Prior to demolishing a building the housebreaker shores the adjoining party walls. In pulling down the remaining walls the débris is allowed to fall into the basement, so that the shores from the basement level to the street are surrounded with old brickbats and mortar. This material holds the rain that falls upon it without, however, becoming really sodden, and as there is no circulation of air around the timbers the lower ends of the shores quickly fall a prey to dry rot. On this account I have had to condemn shores as dangerous structures.

Fig. 44 is some one's idea of shoring a chimney which is in a dangerous condition.

I need hardly deal with dangerous structures from a legal point of view. In London they are governed by Sections 102 to 114, 116 and 117 of the London Building Act, 1894, and Section 5 of the London Building Act, 1894, Amendment Act, 1898.

These sections were referred to by Mr. O. C. Hills in his paper on "The London Building Acts" read before this Institute last Session, and the usual procedure with regard thereto is set out in detail by Mr. G. H. Blagrove in his book on "Dangerous Structures."

Outside London, in localities which have no special Act of their own, the provisions of the Public Health

Fig. 45.

Act, 1875, Section 160, and Section 75 of the Towns Improvement Clauses Act, 1847, would be applicable.

It may be asked, What constitutes a dangerous

structure? I must confess I cannot give a direct reply. It depends entirely upon the circumstances. The whole of the facts that have, or may have, any bearing on each particular case must be examined.

Taking a wall as the simplest case, we must see whether it is out of plumb and how much, whether it is low or high, tied in at frequent intervals by floor joists or beams, or whether there are no ties at all, as is sometimes the case in a flank wall, whether it has lateral support in the shape of return walls or chimney breasts, and if so at what intervals apart, also the general condition of the masonry, whether cracked, bulged, unbonded, built with poor or bad mortar, the nature of the foundation upon which it rests, or which experience in the locality shows it is likely to have or not to have.

Fig. 45 shows the remarkable way in which walls will sometimes refuse to fall, and illustrates the difficulty of applying any theory to the subject.

Conclusion.

I have now enumerated some of the more common causes of dangerous structures, and as only part of the ground has been covered I trust some interesting information may be forthcoming in the discussion.

I must add that I am much indebted to Mr. W. E. Riley, F.R.I.B.A., the Superintending Architect of the London County Council, for his kindness in lending me the photographs shown in Figs. 2, 3, 5, 12, 34, 35, 38, 39, 44, and 45, and to Mr. P. J. Black for the assistance he has rendered in taking the other photographs illustrating the paper.

DISCUSSION.

THE PRESIDENT (Professor Henry Adams, M.Inst.C.E., etc.) :—When the subject was under discussion, the author modestly asserted, that there was nothing he could say that would be of any value, but my experience is that these modest men give the best papers, and we have a sample of it to-night.

I do not remember a paper upon the same subject at any society, and its novelty makes it all the more welcome.

In his opening remarks the author stated that "perhaps few of us have had any experience with failures arising from faulty design and materials in modern structures." Now, it has been my lot to come across a great many failures from faulty design in modern structures—church and other large roofs, from the trusses being badly designed and insufficient allowance made for thrust ; retaining walls, from want of knowledge of principles of stability ; arches, from ignorance of the stresses produced ; girders, from inadequate riveting, cover plates too short, depth insufficient for the span, etc. ; tall chimneys, from insufficient foundations and allowance for wind ; tanks, from badly designed joints ; and many other varieties of error. In fact, where an architect is his own engineer, it is somewhat difficult to find a building wholly free from criticism as regards its structural stability.

The bulging of basement walls below a set-off is due in most cases to the set-off being on one side and consequent upon the eccentricity of the loading, but may be aggravated by the pressure of earth on the other side. On the upper floors the weight of brickwork nearer the outer face, and the floors at the inner face tend to equalize the pressure at an inside set-off.

When a wall fails by breaking in triangular and wedge-shaped portions, it is a clear sign of overloading, and is similar to the failure of crystalline or granular specimens under a compression test. The load on the wall may be within the allowable limit for good material and workmanship, and the failure then takes place from bad mortar, soft bricks, and insufficient bonding.

As regards brickwork running to more than 12 in. for four courses, it should be remembered, that bricks from the Midland Counties are larger than those made in the South of England. The vertical joints inside a wall may be properly flushed up by larrying or by grouting, if carefully done. The condition of the stone piers supporting the dome of St. Paul's Cathedral

shews what may happen, when the interior joints are not filled up solid.

I agree with the author's suggestion, that the lax use of the word "foundation" should be corrected, and that the term "foundation" should mean the artificial construction prepared to support the superstructure, whilst the soil beneath should be termed the foundation bed. I also agree that upon a clay soil the foundations should be deeper than is usual. My practice is to make 5 ft. the minimum, but there is no doubt, that the 8 ft., recommended by the author, would be eminently desirable.

I remember a warehouse, built in the North of London, where the column supports sunk, as described by the author at page 116, but in this case the mortar and concrete were being made of "clean, sharp sand and good ballast" scraped off the top of the permanent way of the adjoining railway.

The author mentions a case, where the top flange of a steel joist was cut away. A year or two ago I had to inspect a case of failure, and found, that the collapse of the building was due to the top flange of some rolled joists being cut away just where cross joists provided the principal load. Although it happened after working hours, a man was killed, whereas if it had happened the next day, the whole Board of Directors would have met the same fate. The chief matter for reflection is, that the building had been up thirty years and on the point of failing all the time.

As regards eaves gutters falling, only this morning coming out of my house I noticed a length of cast-iron eaves gutter, that had fallen in the night, although I have the gutter examined and cleared every autumn.

It would take too long to go into the question of shoring, but I may say this: It is not possible to calculate the thrust, that raking shores have to resist, but it is easy to calculate what any given system of shores is capable of resisting. The limiting factor is generally the weight of brickwork above the needles contained in an inverted equilateral triangle.

In conclusion, I think it will be found, that the prevailing feeling among those, who have often to deal with dangerous structures is, not why there are

so many actual failures, but why there are not ten times the number. Cases are constantly being discovered, where failure ought to have taken place, but a kind Providence seems to have held it back.

MR. A. DOVASTON, of the Dangerous Structures Department of the London County Council :—In speaking in support of the vote of thanks to Mr. Perkins for the very admirable paper, he has read to the meeting to-night, I should like to say on behalf of Mr. W. E. Riley, the head of the department of the London County Council to which I belong, how much he regrets that, owing to a very important engagement that has supervened, he is unable to attend this evening. However, on his behalf, I should like to offer some notes on the subject-matter of Mr. Perkins' paper, which may be of interest to the meeting.

Some time ago I made a comparative analysis of some of the regulations which have been framed from time to time, since the great Fire of 1666, relating to the construction of buildings in London, with the result, that in my view, these Regulations might to some extent account for "dangerous structures" in London.

In the year 1679 an Act was passed for the rebuilding of the City of London, and since that date some fourteen amending Acts have been passed, affecting the construction of the buildings in that and the neighbouring areas. The provisions of the earlier Acts were directed, to secure the use of materials, which offered better resistance to the spread of fire than the old timber-framed buildings existing in 1666, but until 1855 no serious attempt was made either to regulate the manner in which these materials should be put together or their quality as regards durability.

In 1774 an Act was passed, which had for its object the better regulation of "buildings and party walls" and "for the more effectually preventing mischief by fire," not only within the City, but also in certain specified parishes around the City ; it covered an area about 46 square miles in extent and extended for some 4 miles' radius from St. Paul's Cathedral, and this Act remained in force until 1844.

As its title indicates, it dealt to a great extent with this question from a fireman's point of view, and required the external walls or internal enclosures to be of brick, stone, lead, copper, tin, slate, tiles, or iron, and contained an astonishing provision, which allowed the use of what was described as "the necessary planking for the foundation" of the walls.

The provisions of the Act of 1844, which remained in force until 1855, were made applicable to houses within an extended area of 117 square miles, it being practically the same as that of the County of London to-day, and, although it made a step in the direction of obtaining a better form of construction, by specifying the thickness of the brick walls, there were no restrictions whatever as to the quality of the materials or the workmanship.

The Act of 1855 was also weak in these requirements, but it *did* require, that the bricks should be properly bonded and solidly put together and the return walls to be properly bonded together, which is an important provision as regards front walls and parapets of buildings.

By this Act the walls could be erected on a layer of concrete of an unspecified quality and thickness or width, as an alternative to solid ground, which doubtless brought about a weakness in the walls, constructed under this regulation.

By-laws of 1878, embodied in the present Act of 1894, required that bricks for external walls shall be good, hard, sound, and well burnt ; that mortar and cement shall be "good" and of a certain specified composition ; "and that foundations shall be of lime or cement concrete of a specified composition, width, and depth," and it can reasonably be expected, that the rigid enforcement of these provisions should to a large extent minimize "dangerous structure" cases in the future.

Of the 6,000 odd cases of this class, dealt with in 1911, about 66 per cent. arose within the area limited by the 4-mile radius from St. Paul's, and the condemnations have to a large extent arisen owing to the fact of the greater part of that area having been closely built upon before 1878, without being subject to the provisions enacted at that date for

ensuring greater stability and durability, to the disturbing effect of the vibration set up by the heavy motor traffic in recent years on such badly constructed buildings.

MR. W. A. GREEN, M.A., B.Sc.Eng., Assoc.M.Inst.C.E. (Member of Council), said :—Through the kindness of my friends, Messrs. Hall and Owen, I am enabled to give some particulars of a dangerous structure, they were asked to examine, which I think may be of interest to members.

A roof over a certain public building is carried by two lattice girders about 57-ft. span, which support plate girders in the central bay and three light lattice girders in each side bay, the total width of the three bays being about 92 ft.

The roof covering is of asphalte laid on concrete and filler joists, and in addition to this load there are a plaster ceiling and sundry other architectural features, the total weight on one lattice girder being about 100 tons, though it is difficult to estimate this accurately.

The only access to the structure was through a trapdoor, 15 in. × 12 in., whose size possibly accounted for the obvious lack of inspection.

The steelwork has not been painted since erection, more than ten years ago, and is very rusty, a possible reason for this being, that a public meeting in the room below causes the steelwork to run with moisture ; at any rate, this was the case for the few days, it was under observation, before the building was closed to the public.

There are holes in the ends of the filler joists and holes in the top flanges of the lattice girders, but the bolts are conspicuous by their absence. This may not appear a serious omission, but that hardly applies to unfilled rivet holes in diagonal ties of the main lattice girders. In one case one or two rivets were omitted.

The workmanship seems poor throughout. Though brackets are provided for the plate girders, they are in all cases clear of them, the girders being carried by bolts through their end cleats, which are packed out with washers. In several cases the rivets at one end of a diagonal are one fewer in number than those

at the other end—e.g. two at one end and three at the other.

Between the end support and a concentrated load, though the shear could not vary greatly, the rivets in the shear members varied from four (the maximum number in any diagonal member) to two.

The lattice girders have as verticals two $4 \times 3 \times \frac{3}{8}$-in. tees, and as diagonals single $3 \times 3 \times \frac{3}{8}$-in. crossed angles, the booms being two $4 \times 4 \times \frac{1}{2}$-in. L's, with $15 \times \frac{7}{16}$-in. plates. The depth of the girder is 7 ft. $8\frac{1}{2}$ in., and the width of the end panel 5 ft. 6 in. There seems to have been no ambition to bring concentrated loads from the cross girders on to the lattice girders at panel points, and where the outstanding legs of diagonal angles are in the way they are cut off.

Though probably the structure has been dangerous, since the day it was erected, the building was not closed to the public till several days after it was discovered that the end diagonal of one of the lattice girders had broken off short through a rivet hole, with the result that the first vertical had dropped about $3\frac{1}{2}$ in., the compression diagonal being buckled about 10 in. in the form of an S.

A good many of the other diagonal struts are slightly bowed in the companion girder.

For an end reaction of 50 tons the compression and tension in the diagonals, if taken as equal, would be 34 tons, or 16·1 tons per square inch on a gross area of 2·11 square inches.

There are no signs of elongation at the fracture, and the ends are too rusty for a description of its quality to be given. Unfortunately, there seems no prospect of the metal being tested.

The rivet holes (of which there are four of about $\frac{3}{4}$-in. diameter at the ends of the broken member) are punched, and apparently the rivets do not fill the holes.

It would be interesting to know, why the roof stays up, but, unfortunately, promised drawings of the somewhat complicated structure have not been forthcoming, so that I will not waste members' time in speculation.

I, however, wish to call attention to the following remarkable points:—

Consulting engineers had nothing to do with the

roof, though there are reputable engineers responsible for the rest of the structural work in the building.

Some time ago a fresh coat of asphalt was applied. This incidentally added more load to the already overstressed structure, but the surprising thing is that, though I understand there was an obvious hollow in the roof, nobody was sufficiently curious to find out the cause, the only problem being how to drain the hollow without filling up solid with asphalt.

The first point, I wish specially to bring home, is the sin of asking for tenders for an undesigned structure and then placing the order on price without expert opinion on structural adequacy.

The second point is the pity of hiding failures, instead of investigating them in detail and giving the whole profession the benefit of the lessons to be learned thereby.

MR. E. LAWRENCE HALL, Assoc.M.Inst.C.E., M.C.I. :—I have listened to the paper with a considerable amount of interest, but I must confess to being disappointed at one omission, which, however, I hope the author will be kind enough to make good. I refer to the question, " When does a structure become dangerous? "

In the cases illustrated no such question arose, there being no doubt about the danger, but the author must in his experience have met with many cases, in which it was debateable whether a structure should be condemned, and he may have formulated some code of significant symptoms to be looked for in coming to a decision.

From one cause and another failures are constantly happening, but the wonder is that they are so comparatively few in number, considering the liberties taken with buildings by occupiers, who cannot be expected to know better, as well as by builders, and even by architects.

My late partner had an office over a railway terminus, and a gasfitter, who had to do some work there, informed him that he had had no end of difficulty in getting the pipes through, having in one case to drill through *5 in. of solid iron.*

I was called in about sixteen years ago to prescribe

for the roof of Bishopsgate Chapel, which carries a flat ceiling of some extent. This ceiling had sagged about 11 in. I found that a lantern light had been inserted in the roof, and the members of a timber truss had been very badly cut into, with the result that the failure of the roof appeared to be only a question of time. To save the roof, I designed a steel truss with double members, to embrace the injured truss, and provided screw arrangements at the two queen posts, not only to ensure that the new truss should take all the weight, but with a view to reducing the deflection of the ceiling.

The truss was made in pieces and put together in position, and the screwing-up process was commenced while I was out of town, and before long the builder was at my office, terribly worried and excited—the steel truss was *buckling up*.

As I had not only calculated for taking the whole load of the roof, but, knowing the weakness of a workman for turning a screw as long as any power he can command will move it, I had also provided enough strength to overcome the resistance of the timber truss ; it was fairly obvious what had been done to produce such a result as the builder announced, and my partner simply pointed to a note on the drawing, "The truss is to be screwed up very slowly, a turn at one screw and then a turn at the other, alternately." No attention had been paid to this important direction, and hence the trouble.

In the end we succeeded in taking about half the camber out of the ceiling ; any attempt to do more would probably have loosened the plaster.

My architect friends have caused me a little anxiety from time to time. In one case 80 tons of concrete were added to a dome of 110-ft. span ; in another case a 40-ft. arch over a public way had an important abutment omitted ; and in a third case a wall was carried up irregularly on a curved girder. I am glad to say, disaster was averted in each of these cases.

SIXTY-EIGHTH ORDINARY GENERAL MEETING

WEDNESDAY, APRIL 19, 1916

THE SIXTY-EIGHTH ORDINARY GENERAL MEETING of the CONCRETE INSTITUTE was held at Denison House, 296, Vauxhall Bridge Road, Westminster, London, S.W., on Wednesday, April 19, 1916, at 5.30 p.m.

THE PRESIDENT (PROF. HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc.), in the Chair.

The Meeting was devoted to a continuation of the discussion of Mr. W. G. Perkins' paper entitled, "Some examples of Dangerous structures."

SEVENTH ANNUAL GENERAL MEETING

WEDNESDAY, MAY 17, 1916

THE SEVENTH ANNUAL GENERAL MEETING of the CONCRETE INSTITUTE was held at Denison House, 296, Vauxhall Bridge Road, Westminster, S.W., on Wednesday, May 17, 1916 at 5.30 p.m.

THE PRESIDENT (PROFESSOR HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc.), took the Chair.

THE SECRETARY (Mr. H. KEMPTON DYSON) read the notice convening the Meeting.

The following gentlemen were then elected as shewn :—

MEMBER.

ALFRED TOMLINSON, Assoc.M.Inst.C.E., M.Western Australia Inst.Engrs., M.Sc. (Hons.Eng.), Manchester and Western Australia ; Acting Professor of Eng., Univ. of W.A., Perth, Australia.

ASSOCIATE-MEMBERS.

DAHYABHAI BALABHAI KORA, L.C.E. (Bombay), A.M.S.E., M.R.San.I., Assistant State Engineer, Durbargadh, Gondal, India.

FREDERICK CUTHBERT PELHAM LAWRENCE, Superintendent of the Civil Engineering and Structural sections of the Crystal Palace School of Engineering.

THE SECRETARY then read the Report of the Auditors upon the Balance Sheet.

REPORT OF COUNCIL FOR 1915–16 SESSION.

The present Membership of the Concrete Institute and the alteration since the previous figures were given in the Report for the 1914–15 Session is shown in the following table :—

NUMBER OF MEMBERS.

	End of Dec., 1914.	End of Dec., 1915.	April :oth, 1916.
Members	937	880	877
Associate-Members	32	43	46
Associates	6	7	7
Students	56	57	57
Special Subscribers	5	5	5
Hon. Members	16	10	10
Total Membership	1,052	1,002	1,002

Of this total 360 reside in London and its environs, 381 reside in the provinces, and 261 abroad.

It is a matter for congratulation that there has not been considerable decrease in Membership owing to the War, although a number of Members of the Institute are serving with the Forces.

The finances of the Institute are shown by the accompanying Balance Sheet. In 1913, 1914, and 1915 there were deficits. A greater number of subscriptions are in arrear than in normal times. Rigid economy has been exercised, and the printing of the TRANSACTIONS has been delayed. It is hoped that when peace is declared and business has resumed its normal course, the past rate of growth of the Institute in Members and income will be resumed.

The number of general meetings has been reduced, and in view of many of the junior Members joining the Forces the Educational Lectures and the Informal Meetings of Junior Members have again been abandoned for the Session; such educational work will be resumed in the future.

The following is a list of Meetings during the Session :—

1915.

Wednesday, November 17th. Sixty-fourth Ordinary General Meeting.
Second Presidential Address by Professor Henry Adams, M.Inst.C.E., M.I.Mech.E., F.S.I., M.S.A., etc.

1916.

Wednesday, January 19th. Sixty-fifth Ordinary General Meeting.
Discussion on Report of the Joint Committee on Loads on Highway Bridges.

Wednesday, February 16th. Sixty-sixth Ordinary General Meeting.
Paper by Mr. Charles F. Marsh, M.Inst.C.E., etc., entitled "Reinforced Concrete as applied to Waterworks Construction."

Wednesday, March 15th. Sixty-seventh Ordinary General Meeting.
Paper by Mr. W. G. Perkins, District Surveyor for Holborn, entitled "Some Examples of Dangerous Structures."

Wednesday, April 19th. Sixty-eighth Ordinary General Meeting.
Continuation of Discussion on Paper by Mr. W. G. Perkins, District Surveyor for Holborn, entitled "Some Examples of Dangerous Structures."

Wednesday, May 17th. Seventh Annual General Meeting.
Paper by Dr. Oscar Faber, D.Sc., Assoc.M.Inst.C.E., A.C.G.I., M.C.I., etc., entitled "Shear in Reinforced Concrete Beams."

The thanks of the Institute are due and are hereby tendered to the authors of papers.

The attendance at meetings during the past Session has been well maintained.

As the result of a ballot among Members of Council, the bronze medal for the best paper read in the 1914-15 Session has been awarded to Mr. F. E. Wentworth-Sheilds, M.Inst.C.E., etc., for the paper entitled "The Design of Quay Walls."

The Regulations made under the provision of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete, to the amendment of which the Council and Committees of the Institute have given much attention in former years came into force on January 1, 1916.

The Institute was asked by the Advisory Council appointed by the Committee of the Privy Council for Scientific and Industrial Research to co-operate by furnishing a statement indicating the specific problems, requiring scientific investigation, of the industries with which this Institute is most intimately concerned. The matter was considered by the Science Standing Committee and the Council who submitted a list of subjects for research, and in the covering letter it was pointed out that the research work in this country, upon the design and construction of structures, more particularly those of concrete, reinforced concrete and steel, had been sporadic and of minor value to practical work, because, quite apart from lack of funds for such work, there had been lack of co-ordination and control by those who would be engaged in the practical application of the knowledge sought from such research. For instance there had been very few tests of real utility to designers of reinforced concrete, though that was one of the new modes of construction whose application had been widely extended of recent years. Most of the information available to designers had been derived from experiments conducted in France, the United States, and Germany, but seeing that such experiments had largely been framed in view of special local conditions, they did not generally directly apply to practice in this country.

After reference to the occupation and numbers of

Members of the Institute, the offer was made of the services which it was hoped the Advisory Council would require of the Institute, namely, advising in detail on tests of practical utility to the Structural Engineer; in addition the offer was made of any assistance in the way of supervision of the ways and means of research which the Advisory Council might entrust to the Institute's Council. In reply the Research Council invited the representatives of the Institute to meet their Chairman, Sir William M'Cormick, and the following attended: Professor Henry Adams, President C.I., M.Inst.C.E., etc., Mr. E. P. Wells, J.P., Past President C.I., Mr. H. D. Searles-Wood, Vice-Pres. C.I., F.R.I.B.A., Mr. R. H. Harry Stanger, Assoc.M.Inst.C.E., and the Secretary, Mr. H. Kempton Dyson. The Institute is now engaged in formulating detailed particulars in respect to an application for a grant in aid of researches that can be immediately undertaken by various laboratories and engineering institutions on a co-ordinated scheme.

The Council of the Concrete Institute have appointed the following as a Special Committee to consider and report on the relations of the Architect and the Reinforced Concrete Specialist and Structural Engineer:—Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A., Prof. Henry Adams, M.Inst.C.E., Pres. C.I. (ex officio), Dr. Oscar Faber, D.Sc., Assoc.M.Inst.C.E., Mr. H. D. Searles-Wood, F.R.I.B.A., Vice-Pres. C.I., Mr. T. B. Shore, Mr. R. W. Vawdrey, B.A., Assoc.M.Inst.C.E., and Mr. H. Kempton Dyson (Hon. Secretary).

The Committees appointed by the Council for the Session were as follows:—

The Finance and General Purposes Committee.

Chairman.—Mr. H. D. Searles-Wood.

Ordinary Members.—Professor Henry Adams, Mr. S. Bylander, Mr. E. Fiander Etchells, Mr. J. E. Franck, Mr. Charles F. Marsh, Sir Henry Tanner, Mr. H. J. Tingle, Mr. E. P. Wells, and Mr. G. C. Workman.

Ex officio—The President and the Chairman of each Standing Committee.

THE SCIENCE STANDING COMMITTEE.

Chairman.—Mr. H. D. Searles-Wood.
Vice-Chairman.—Mr. W. G. Perkins.
Hon. Secretary.—Mr. M. E. Yeatman.
Ordinary Members.—Professor Henry Adams, Mr. Ewart S. Andrews, Mr. H. K. G. Bamber, Professor T. Hudson-Beare, Mr. D. B. Butler, Mr. E. Fiander Etchells, Dr. O. Faber, Mr. J. E. Franck, Mr. M. Garbutt, Mr. H. C. Johnson, Mr. Charles F. Marsh, Dr. J. S. Owens, Mr. A. R. Sage, Mr. R. H. Harry Stanger, Mr. R. W. Vawdrey, Mr. E. P. Wells, and Mr. F. E. Wentworth-Sheilds.

THE REINFORCED CONCRETE PRACTICE STANDING COMMITTEE.

Chairman.—Mr. S. Bylander.
Vice-Chairman.—Mr. G. C. Workman.
Hon. Secretary.—Mr. R. W. Vawdrey.
Ordinary Members.—Professor Henry Adams, Mr. Ewart S. Andrews, Mr. Percy J. Black, Mr. J. F. Butler, Dr. O. Faber, Mr. J. Petrie, Mr. F. Purton, Mr. Lewis H. Rugg, Mr. A. Alban H. Scott, Mr. H. D. Searles-Wood, Mr. T. B. Shore, Mr. B. Taylor, Mr. J. M. Theobald, Mr. T. A. Watson, Mr. E. P. Wells, and Mr. M. E. Yeatman.

THE PARLIAMENTARY STANDING COMMITTEE.

Chairman.—Mr. J. Ernest Franck.
Vice-Chairman.—Mr. Osborn C. Hills.
Hon. Secretary.—Mr. P. J. Black.
Ordinary Members.—Professor Henry Adams, Mr. Matt Garbutt, Mr. H. Percy Boulnois, Mr. W. G. Perkins, Mr. W. E. A. Brown, Mr. E. Fiander Etchells, Mr. E. O. Sachs, Mr. E. P. Wells, and Mr. G. C. Workman.

The Members of Council chosen to retire under the Rules of the Institute were as follows :—

Mr. H. K. G. Bamber.	Mr. Alexander Drew.
Mr. A. C. Davis.	Mr. L. Serraillier.

Of these Messrs. Bamber and Serraillier were eligible for re-election, and their names have been submitted to the Members.

In addition the Council nominated—

Professor J. D. Cormack. Mr. Edgar H. Homan.
Mr. W. A. Green. Mr. Bertram L. Hurst.

The District Surveyors' Association asked the Institute to nominate representatives upon a Conference on the working of the London County Council (General Powers) Act, 1909, with reference to steel frame buildings. The Members of the Conference are as follows—

Representatives of the Royal Institute of British Architects

Mr. F. R. Farrow, F.R.I.B.A. (*Chairman*).
Mr. W. E. Vernon Crompton, F.R.I.B.A.
Mr. F. N. Jackson, Assoc.M.Inst.C.E., Hon. A.R.I.B.A., M.C.I.

Representatives of the District Surveyors' Association—

Mr. Arthur Ashbridge, F.R.I.B.A.
Mr. Bernard J. Dicksee, F.R.I.B.A.
Mr. E. Alexander Young, A.R.I.B.A.

Representatives of the Concrete Institute—

Mr. S. Bylander, M.C.I.
Mr. E. Fiander Etchells, Assoc.M.Inst.C.E., Hon. A.R.I.B.A., F.Phys.Soc., M.C.I., etc.
Mr. W. G. Perkins, M.C.I., District Surveyor for Holborn.

Hon. Secretary

Mr. H. Kempton Dyson, Secretary C.I.

The Rules and Syllabus of the proposed examination of the Concrete Institute were appended to a previous Report of the Council. The first examination is deferred until after the War.

The Council regrets to record the decease of :—

Killed in Action—

Lieut. J. Hopkinson,
3 Whitehall Court,
London, S.W.

INCOME AND EXPENDITURE ACCOUNT.

Year Ending December 31, 1915.

Dr. | *Cr.*

INCOME.

	£ s. d.	£ s. d.
To Annual Subscriptions		790 6 7
,, Special Subscriptions		26 5 0
,, Students' Subscriptions		13 2 6
,, Entrance Fees		30 9 0
,, Sundry Receipts :—		
Various Recipts	5 1 0	
Interest on Deposit	2 17 10	
Advertisements	12 11 1	
Donations	1 1 0	
		21 10 11
,, Balance carried down		13 18 4
		£895 12 4

EXPENDITURE.

	£ s. d.
By Office Salaries	359 15 0
,, Office Rent, Hire of Hall, Cleaning, Lighting, Heating ...	147 17 9
,, Printing, Reporting, &c.	167 2 1
,, Postage, Telegrams, &c.	41 15 1
,, General Expenses	52 11 7
,, Accountancy	8 8 0
,, Stationery	49 5 7
,, Books and Utensils	2 16 5
,, Legal Expenses	48 19 10
,, Amount written off Furniture for Depreciation	14 1 0
	£895 12 4

	£ s. d.
,, Balance, being excess of Expenditure over Income 12 months to date, transferred to Surplus Account...	13 18 4
	£13 18 4

	£ s. d.
,, Balance brought down	13 18 4
	£13 18 4

BALANCE SHEET.

Year Ending December 31, 1915.

Dr. | *Cr.*

LIABILITIES.	£ s. d.
To Current Liabilities	172 12 0
„ Life Members' Subscriptions	42 0 0
„ Subscriptions received in advance	30 18 0
	£245 10 0

ASSETS.	£ s. d.	£ s. d.
By FURNITURE :—		
As at December 31, 1914	138 2 0	
Additions—12 months	2 19 0	
	141 1 0	
Less Depreciation...	14 1 0	
		127 0 0
„ CASH :—		
At Banker's, Current Account	53 16 6	
In hand, Subscriptions, &c.	1 0 7	
Petty Cash	10 8 6	
		65 5 7
„ Balance, being excess of Expenditure over Income as at 31st December, 1914 ...	39 6 1	
Add Balance, being excess of Expenditure over Income 12 months to date, as per account	13 18 4	
		53 4 5
		£245 10 0

We report to the Members that we have obtained all the information and explanations we have required, and that we have examined the above Balance Sheet dated 31st December, 1915, with the Books and Vouchers of the Concrete Institute. We certify that such Balance Sheet is properly drawn up so as to exhibit a true and correct view of the state of the Institute's affairs according to the best of our information and the explanations given us, and as shown by such Books and Accounts.

(*Signed*) MONKHOUSE, STONEHAM & CO.,
Chartered Accountants.
SALISBURY HOUSE,
LONDON, E.C.
13th *April*, 1916.

FOR THE CONCRETE INSTITUTE :
(*Signed*) HENRY ADAMS, *President.*
E. FIANDER ETCHELLS, *Chairman of Finance Committee.*
H. KEMPTON DYSON, *Secretary.*

Several donations to the Library have been received by the Council from authors, publishers, and kindred societies, and the Council expresses thanks to the donors. A list of books received is published from time to time in the TRANSACTIONS.

Finance and General Purposes Committee.

The Finance and General Purposes Committee has held regular meetings preliminary to each Council Meeting, and the general results of their deliberations are contained in the foregoing particulars of the Council's work for the year.

Science Standing Committee.

As the outcome of a resolution of the Joint Meeting of the Science and Tests Standing Committees the Council decided to combine the two Committees under the name of the Science Committee. The appointment of the Members of the separate Committees was therefore rescinded, and those whose names are recorded in the foregoing list were then appointed Members of the newly constituted Science Committee. The Committee has formulated a list of subjects for research which has been referred to above under the work of the Council, and has begun to consider the various suggestions that have been made for the amendment of the Building Acts throughout the country.

The Committee has answered various inquiries made by members for advice.

The Science Standing Committee has the following matters under consideration :—

1. Amendment of the London Building Acts and Building By-laws generally.
2. Standardization of joints and connections in reinforced concrete.
3. Amendment of the Standard Specification for cement.
4. Co-ordination of the Standard Specifications for structural steel of all kinds.

Reinforced Concrete Practice Standing Committee.

This Committee has not held any further joint meetings with delegates of the Quantity Surveyors' Associa-

tion in reference to the Report on a Standard Method of Measurement for Reinforced Concrete except to revise the report in view of the discussion and get it finally approved as regards building construction by the Councils of the two Societies concerned. The first part of the Report, namely, that dealing with building construction, has been published by the Quantity Surveyors' Association, but the Concrete Institute is deferring publication of the Report until it has been completed by a section dealing with Quantities for Engineering Work.

The Committee has in hand the drafting of a Report on the Supervision of Reinforced Concrete Work containing recommendations which it is to be hoped will be helpful to inspectors, clerks of works, and foremen as regards the execution.

The Reinforced Concrete Practice Standing Committee has the following matters under consideration :-

1. Advice to clerks of works, inspectors, and foremen as to methods of properly executing concrete work and of preventing defects and failures.
2. Regulations, recommendations of joint committees, and various methods of calculation in respect to the design of reinforced concrete and the like.
3. Standard concrete mixtures for general purposes.
4. The use of cinder, ash, clinker, and breeze in concrete.
5. Methods of making concrete watertight and of waterproofing concrete.

Parliamentary Standing Committee.

The Parliamentary Standing Committee has the following matter under consideration :

The draft of a Bill promoted by the Society of Architects for the registration of architects.

Investigation Committee.

The Investigation Committee has had under consideration reports of a failure of a reinforced concrete structure, but as the information contributed was

confidential, the results of their deliberations cannot be furnished in the form of a Report.

Joint Committee on Loads on Highway Bridges.

The Joint Committee on Loads on Highway Bridges convened by the Institute submitted their draft Report for discussion at a General Meeting.

It is completed, and will shortly be issued in separate form.

The reception of the Report was proposed by the outgoing President, and was duly seconded and carried. The Report of the Scrutineers on the annual election of four Members of Council was read by him, after which the re-appointment of Messrs. Monkhouse, Stoneham & Co., as the Auditors for the ensuing year at a fee of five guineas was moved, seconded, and carried.

The next business was to present the Institute's Medal for the best paper read during the Session 1914–15, to Mr. F. E. Wentworth - Sheilds, M.Inst.C.E., V.P., for his paper, entitled "The stability of Quay walls on Earth foundations"; the President made the presentation in a few well-chosen remarks, to which Mr. Wentworth-Sheilds suitably responded.

The President then vacated the Chair and installed Mr. Wentworth-Sheilds as President for the ensuing two years. The new President acknowledged the kind remarks, made by Professor Adams, and moved a hearty vote of thanks to the Professor for his very able occupancy of the Chair during the past two years, a period which had witnessed a great world-upheaval, whose end was not yet. Professor Adams thanked the Meeting for its very cordial reception of the vote, so generously proposed by Mr. Wentworth-Sheilds, who then called upon Dr. Oscar Faber, D.Sc. (Lond.), Assoc.M.Inst.C.E., A.M.I.E.E., A.C.G.I., who read the following paper :—

SHEARING RESISTANCE OF REINFORCED CONCRETE BEAMS

(*a*) STATEMENT OF ORDINARY THEORY AND FORMULÆ.*

1. Resistance of concrete to diagonal tension.

Consider a small square element in the web of a beam subjected to shear (Fig. 14).†

The vertical shear in the beam produces shearing stresses S_1 on vertical planes. If these were the only forces acting on the element it would rotate. To keep it in equilibrium there must be an equal shear stress, S_2, on the horizontal planes.

If these shear stresses are combined, it will be seen that they are equivalent to a principal compressive stress on one diagonal plane and a principal tensile stress on the other, both stresses being equal in intensity to the shear stress.

Since concrete is far weaker in tension than in either compression or shear, it tends to fail along the tension plane, and the limiting value for the shear resistance is then—

$$S = t\,b\,a$$

where t is the safe tensile strength of the concrete generally taken as 60 lbs. in.^{2}, b and a are the breadth and the arm

* This is a very short precis only. More complete arguments are given in Faber and Bowie's "Reinforced Concrete Design," pages 79–88.

† The figure numbers refer to the figures in a larger work of which this paper is a part, and are not all reproduced here, though they will be shown on the screen, and are all reproduced in "Concrete and Constructional Engineering" for May 1910 and following numbers.

of the resisting couple in the beam respectively. The formula may therefore be written—

$$S = 60\ b\ a$$

The reason for taking the maximum stress over the area $b\ a$ and not over the area $b\ d$ is that the shear on vertical planes is not constant but varies as a parabola from the top down to the neutral axis, as shown in Fig. 15. Remembering that the area of a parabola is two-thirds of the rectangle enclosing it, it will easily be seen that the area of the shear stress curve above the neutral axis is the same as if the shear stress were taken up at its maximum value to the centre of compression, as shown dotted in Fig. 15.

It has recently been felt that there is some theoretical objection to considering the tensile stress as acting on the whole area $b\ a$, since the elongation which must take place before the steel can take up its stress forces the concrete to crack along lines as shown in Fig. 18. which theory indicates must extend to near the neutral axis, even if they are not visible.

It is obvious that once these cracks have been produced by the action of the reinforcement resisting what may be termed flange stresses, it is not permissible to consider the shear as resisted by tensile stresses across such surfaces.

This was undoubtedly the reason for the following regulation in the draft dated 23/7/14 of the L.C.C. Regulations :—

> "66.—The vertical shear taken by the concrete only shall be calculated on the compressed area of the web, or on the web area for a depth equal to one-half of the effective depth of the beam or slab. The intensity of the shearing stress shall not be greater than the values given in Regulation 42."

The practical effect of this is to reduce the safe shear resistance, when no web reinforcement is provided, to $30\ b\ a$ or less, which is much lower than has been required by any other regulations.

It will be seen later that in the writer's view this value may often be exceeded with perfect safety.

In the final regulation this draft regulation has however been amended back to the whole area $b\ a$ (Regulation 64).

2. **Resistance of bent-up bars.**

When part of the main reinforcement is bent up near the end of a beam as in Fig. 16, it will cut the planes of diagonal tension, by which failure will otherwise occur.

It is assumed in common practice that the bar may be considered as stressed up to its safe stress, in which case, of course, its resistance to vertical shear is—

$$S = t\,A \sin\theta$$

where t is the safe tension stress (16,000),
A is the area of inclined bar,
θ its inclination to the horizontal.

3. **Resistance of stirrups.***

The action of stirrups in resisting shear in a beam is to be understood by considering them as the vertical tension members of a lattice girder, the diagonal compression member being formed by the concrete in the web of the beam (see Fig. 17).

Consider first the case when the spacing of the stirrups l is equal to the arm of the resisting couple a, and the diagonals are inclined at

$$\theta = 45^\circ$$

It is obvious that under these conditions the tension to be resisted by each stirrup is equal to the total shear across the the section (or such proportion of it as is to be carried by the system of stirrups).

Suppose now that the stirrups are placed twice as close. The stress in the stirrups will depend on whether we consider the system as a single one with the inclination of the diagonals doubled, as in Fig. 19 (*a*), or as a double one with the inclination at 45°, as before as in Fig. 19 (*b*).

In the former case it will be seen that the tension per stirrup is still equal to the total shear.

* The writer believes that this is a rather generous exposition of the ordinary theory as generally understood. Note, for example, Appendix II to the R.I.B.A. (1911) Report, which is based on the *shearing* strength of the stirrups, whereas it has been proved experimentally (and is clear from theoretical considerations) that their action is in tension, and that if put into shear they would crush the concrete on their edge before their strength were developed.

In common practice it is assumed that the inclination will remain at 45°, in which case the safe resistance to shear increases in proportion to the closeness of the stirrups, or, if we put

A = area per stirrup,
l = spacing,
a = radius arm of resisting couple,
t = safe tensile stress in stirrups,

the safe shear resisted by stirrups is—

$$S = \frac{A\,t\,a}{l}$$

A proviso is made that the spacing l must not exceed the radius arm.

4. Combination of 1, 2, and 3.

In most regulations, but not in draft L.C.C. Regulations, and to the writer's certain knowledge in the practice of most so-called reinforced concrete specialists, it is customary to deal with combinations of 1, 2, and 3 by the process of simple addition, which certainly has the merit of simplicity, and perhaps that alone.

For example, in a beam containing stirrups and bent-up bars, the calculations take some such form as the following :—

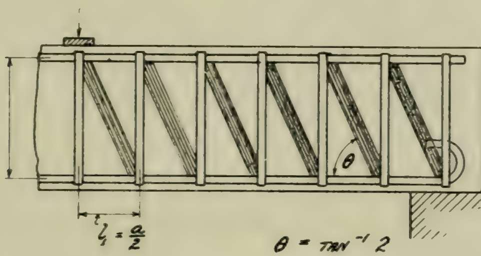
	Shear Resistance.
By concrete	$60\,b\,a$
,, bent-up bars	$16{,}000 \times A \sin\theta$
,, stirrups	$16{,}000 \times \frac{A \times a}{l}$
Total	

and provided this total shear resistance is at least equal to the shear across the section, conditions of safety are supposed to be satisfied.

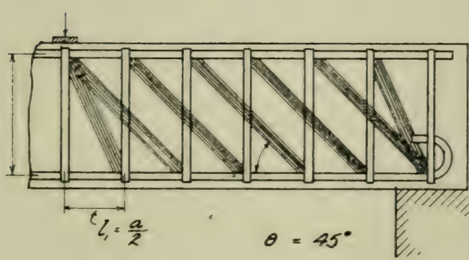
Note in this connection the following extract from page 14 of the Second Report (1911) of the Joint Committee of the Royal Institution of British Architects, which has been used as a standard for several years :—

"When S, the total shear in lbs. at a vertical section, does not exceed 60 $b\,a$, no shear reinforcement is required.

"When S exceeds 60 $b\,a$, vertical shear members may be



(a) CLOSE STIRRUPS ACTING AS SINGLE SYSTEM



(b) CLOSE STIRRUPS ACTING AS DOUBLE SYSTEM

FIG. 19

provided *to take the excess*, and proportioned by the following rule :—

$$\frac{A_s S_s a}{p} = S - 60\,b\,a$$

$$A_s = \frac{(S - 60\,b\,a)p}{a\,s_s}$$

where S_s is the unit resistance of the steel *to shearing* and p is the pitch or distance apart of the vertical shear members or groups of shear members, of area A_s."

It will be seen that besides adding the resistance of the concrete to that of the shear reinforcement, the efficiency of the stirrups is considered to be limited by their resistance to shearing.

The writer hopes to show later that the first is quite wrong, and that the action of the stirrups is one of tension, and not of shearing.

(*b*) STATEMENT OF THE WRITER'S THEORY.

1. Resistance of concrete to diagonal tension.

The writer is in general agreement with the analysis just given under this head, and believes that in very long beams with no shear reinforcement the safe shear across a vertical section may be limited by diagonal tension, in which case the formulæ given are applicable.

In nearly all practical cases, however, the writer believes that a much greater resistance to shear will be provided by the formation of a "direct inclined compression" as explained later.

2. Resistance of bent-up bars.

Similarly with inclined bars the writer approves the use of the method outlined, provided certain precautions are taken. The most important of these is that the bent-up bar shall be adequately fixed at both ends to prevent slipping. It is not difficult to see that for the formula given to be correct the assumed stress must actually exist in the bar for the whole of its inclined length.

Consider, for example, Fig. 20, showing two typical distributions of tensile stress in bent-up bars.

In the top one (*a*) the efficient hook at the end develops a stress of, say, 8,000 in the bar, and this is gradually increased by adhesion to 12,000 along the horizontal portion. The friction at the bend—similar to that of a belt round a pulley—will increase this to the safe stress of 16,000, which is maintained through its inclined length.

At this point its stress will be determined by considerations of bending moments and the friction on the bend will enable it to alter its stress here if necessary. As a rule, no great

change is necessary here, as the bar is usually bent up where the bending moment diagram allows of it.

Consider now the bottom one (*b*) the stress at the top end is necessarily zero, and it can only increase gradually as determined by adhesion. In most practical cases it will not even have reached t = 16,000 at the lower end, but perhaps only 8,000, depending on its length and diameter.

In that case the resistance of the bar to shear should only be taken on a basis of t = 8,000 if we are considering failure on the surface 1, or t = zero, if we are considering failure on the surface 2.

If some apology is needed for the length of this statement, the writer offers the fact that many designers do not appreciate this point sufficiently, and quite often—as, for example, with the prongs of a well-known patent bar which provide no fixing at all.

The writer also feels that some limit must be put to the resistance to shear which can be obtained in this way, since the resultant of the tension in the inclined and horizontal portions of the bent-up bar can only be equilibrated by an inclined compression force, and a limit will be reached when failure or bursting of the concrete resisting this force will occur before the steel is overstressed.

This applies also to the hook or bend at the top end of the bent-up bar.

This limit will be discussed later.

The writer would also add that whereas in common practice and in all regulations with which he is acquainted the resistances to shear of bent-up bars is limited to the length of beam in which the bar is inclined, in his opinion the inclined compression produced by the resultant of the tensions of inclined and horizontal portions of the bent-up bar, give a resistance to shear in the adjacent panels which much increases the value of such an arrangement of reinforcement. This also will be dealt with more fully later.

3. With **stirrups**, the writer agrees the substantial correctness of the usual theory and formula as explained, provided certain precautions are taken. For exactly the same reason as explained above, the stirrups must be adequately secured both ends ; otherwise, if fixed at the bottom and not at the top—a common arrangement—the stress will be zero at the top, gradually increasing towards the bottom at a rate determined by adhesion, very different from the constant stress of 16,000 assumed by the formula.

In the matter of the all-important angle of θ of the diagonal forces with the horizontal, the writer is not satisfied with the tacit assumption of 45° in all cases.

In his view, the following considerations will govern this angle :—

It will have been noted in (*a*) 3, that the smaller this angle the greater is the efficiency of a given arrangement of stirrups in resisting shear. The writer takes the view that

In a complex combination of members which may act in several ways failure will not occur until the members act in such a manner as to produce the maximum resistance to failure consistent with absence of overstrain in any member.

If this general theory be accepted—and in practice it is tacitly assumed in all design, though the writer has not seen it specifically stated—it will be obvious that where the resistance to shear is limited by the cohesion of the stirrups, the angle θ will become as small as is "*consistent with absence of overstrain in any member.*"

The two dangers attaching to the assumption of a small value of θ are—

(1) Overstressing the concrete in compression.
(2) Causing sliding of the stirrups.

The first of these is considered later, where it is shown that C increases in proportion to $\frac{1}{\sin\theta \times \cos\theta}$, and its value may be kept within safe limits provided the total shear across the section does not exceed C $\sin\theta \times \cos\theta$ in favourable cases of distribution.

Considering now the second, reference to Fig. 21 (*a*) will show that a bent-up bar of suitable design, even when vertical, is not subject to any slipping, whereas a stirrup, as in Fig. 21 (*b*), is.

Resolving the inclined compression C horizontally and vertically where it meets the bar, the vertical component P will be resisted by the stirrup, and the horizontal component is resisted by adhesion and friction.

$$\text{P} \cot\theta = \mu \text{P} + \text{adhesion}$$

The adhesion is to be taken to include a horizontal compression in the concrete resisted by the hook at the end.

If, however, we neglect this for a moment we have—

$$\mu = \cot\theta$$

If we take the coefficient of an iron rod on concrete as 0·5 (it will be more for a rusty bar and less for a smooth bar) we have—

$$\theta = \cot^{-1}\tfrac{1}{2}$$

In other words, so long as θ is over $\cot^{-1}\frac{1}{2}$ (about 63·5°) there can be no question of slipping. At smaller angles, however, we are relying on adhesion to the extent of

$$\text{Adhesion} = P\,(\cot\theta - \tfrac{1}{2})$$

Consider the case of a beam in which we have $\theta = 45°$ two bottom bars 1 in. diam., in a beam having the radius arm of the resisting couple 12 in. and the width 5 in., stirrups 4 strands 1 in. $\times \frac{1}{4}$ in. at 12 in. centres.

If we stress the stirrups to 16,000 we have—

$$P = 16{,}000$$
$$\cot\theta = 1$$

Hence—

$$\text{adhesion} = 16{,}000\,(1 - \tfrac{1}{2}) = 8{,}000 \text{ lbs.}$$

Now the adhesion which can safely be developed in a length of 12 in. by two 1-in. bars is—

$$12 \text{ in.} \times 2 \text{ in.} \times \pi \times 100 = 7{,}530 \text{ lbs.}$$

so that in this case the angle of 45° is just about safe.

It may be noted that 4 strands 1 in. $\times \frac{1}{4}$ in. is very heavy shear reinforcement for a 12 in. $\times$ 5 in. beam, the shear across the section being

$$\frac{16{,}000}{12 \times 5} = 266 \text{ lbs./in.}^2$$

whereas 300 lbs./in.^{2} is shown to be the absolute maximum for stirrups.

We may therefore conclude that the angle $\theta = 45°$ is safe in most practical cases.

4. It is chiefly in connection with the calculations for **combinations of the three preceding systems** that the writer takes exception to what is unfortunately too common practice.

It is obvious that the tension in bent-up bars cannot approach a figure of 16,000 lbs./in.^{2} without the corresponding strain, and at that strain the concrete will necessarily have cracked.

This will be realized when we consider that, retaining 15 as the modular ratio, the tensile stress in the concrete would otherwise approximate to

$$t = \frac{16,000}{15} = 1,066 \text{ lbs./in.}^2$$

whereas its ultimate stress (in tension) is about 240, and its safe stress about 60.

It clearly follows that if we stress the steel to 16,000 the resistance of the concrete must be neglected, and if we rely on the concrete we must only stress the steel to

$$60 \times 15 = 900 \text{ lbs./in.}^2$$

This has long been realized in the calculations for resisting moments, but is imperfectly realized in those for shear.

For exactly similar reasons the resistance to shear due to stirrups stressed to 16,000 must not be added to that due to concrete stressed to 60 lbs./in., since the former condition requires the concrete to be cracked.

In both cases the cracks may be so small that they cannot be detected, as is the case in regard to bending, when the concrete can be proved to be cracked long before the cracks can be seen.

Coming now to a combination of stirrups and bent-up bars, there is no reason why they should not be effective simultaneously. Consider, for example, the arrangement of reinforcement, and the stresses induced in it by shear, shown in Fig. 22.

It will be seen that the opening of cracks on diagonal surfaces 1 and 2 does not in any way prevent the concrete from resisting the diagonal compression forces required to bring the stirrups into tension, while they are a necessary condition for the bent-up bars to be usefully stressed.

5. "**Inclined compressions.**"

A few years ago the writer realized that many beams would have a source of resistance to shear which is not included under either of the heads 1 to 3 (concrete in tension, bent-up bars, and stirrups), and to which he has given the name the "inclined compression" resistance. It was

indicated in "Reinforced Concrete Design," pages 83 and 85, together with the conditions for its action and its limitations, but the matter was not dealt with sufficiently fully to be of much assistance to designers, nor was experimental evidence given of its correctness.

Since then the writer has had time to investigate it more fully, both analytically and experimentally, and he may, perhaps, be allowed to mention that the treatment suggested has for some years been used in his drawing office in connection with important structures.

It will have been noticed that in the preceding pages, suggestions have been made that inclined compressions must be produced in the concrete wherever bent-up bars or stirrups are called upon to resist shear, and in fact the only analysis of the action of stirrups which is worth serious study is one which is based on a study of this action.

It will be shown, first, that such inclined compressions sometimes occur where neither bent-up bars nor stirrups occur, and calculations will be made to show the maximum shear which can be resisted in this manner.

We will then make a further study of inclined compressions in combination with bent-up bars and stirrups, which it is hoped will bring fresh light on the subject of resistance of beams to shear.

It will be convenient to refer to inclined compressions as "direct" when they pass from load to support without being lifted by tension members, and as "indirect" when they pass from the load to the bottom of a tension member which lifts them to the top flange, whence they again pass down towards the support, this lifting occurring as many times as may be necessary.

6. Direct inclined compressions.

Consider Fig. 23 (a) which shows a rectangular beam reinforced with straight tension bars with large hooks at the end and loaded with a central point load. Unless we are to overstress our material at the load and the supports, an appreciable area must be given to these, which is indicated in the drawing.

When the load is applied, tensile stresses are produced in diagonal planes, and at a certain stage (corresponding to a tensile shear)

$$= \frac{S}{b\,a} = \text{about } 240 \text{ lbs. in.}$$

a crack will be formed on some surface (like Fig. 22) whether they are visible to the eye or not.

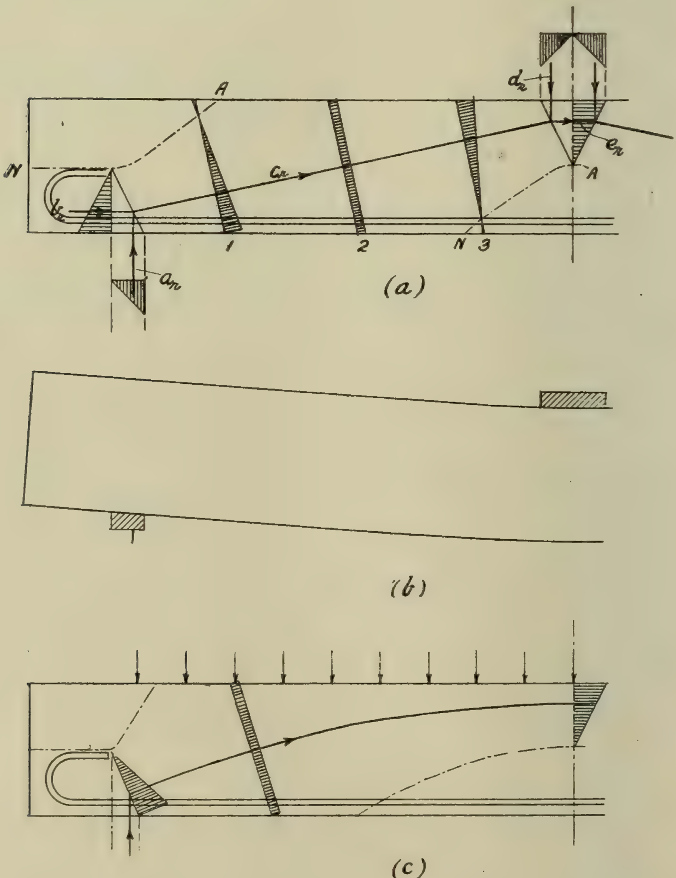


FIG 23

TO ILLUSTRATE THE ACTION OF "DIRECT INCLINED COMPRESSIONS"

The conditions in the web then change completely and approximate to those which it is attempted to indicate graphically in Fig. 23 (a).

The tensile stress in the steel, instead of varying uniformly from a max. at the centre to zero at the support, will tend to become more constant along its length and to die off in a much shorter length near the bearings (see Fig. 24).

The adhesion, instead of being distributed uniformly along the bar from mid-span to support, will tend to become very small during this length, and will be taken up rapidly at the end by the pressure of the hook against the concrete.

The exact conformation of the forces will, of course, depend largely on external conditions. In Fig. 23 (*a*) it has been attempted to show the forces produced when the stress on the support varies from a maximum at the inside edge to zero at the outer edge, as might occur owing to the deflection of the beam, as shown in Fig. 23 (*b*). The compression from the hook is taken to vary from a maximum at the lower surface to zero about half-way up the beam, the compression at mid-span is taken to vary from maximum at the top to zero at the neutral axis, while the distribution at the load is taken to be maximum at the two edges and zero at the centre, a condition which may also be approximated to as in Fig. 23 (*b*).

It is helpful to trace a small force a_n from the support, to where it meets a force b_n from the hook, is resolved into e_n till it reaches the support, where it is again resolved into a load d_n and a horizontal compression e_n.

Only the force a_n, the resultant, has been treated in this manner on the drawing.

If we consider sections 1, 2, and 3 across the resultant inclined compression and consider the stress as uniformly varying across the section, the stress distribution may be somewhat as shown.

It will then be found that the neutral axis, instead of remaining a straight line, now runs up to the top surface near the support, and reappears at the bottom surface near the load, somewhat as shown in Fig. 23 (*a*) and (*c*).

In Fig. 23 (*b*) a similar diagram is drawn for the case of a beam uniformly loaded, in which case the inclined compression is parabolic.

Some will have difficulty in understanding how the conditions described can exist. A common difficulty is found in the fact that the steel is in tension throughout its length while the concrete adjacent to it is in compression having a considerable horizontal component.

This difficulty is due to having always been taught that the stresses of steel and adjacent concrete are of the same sign

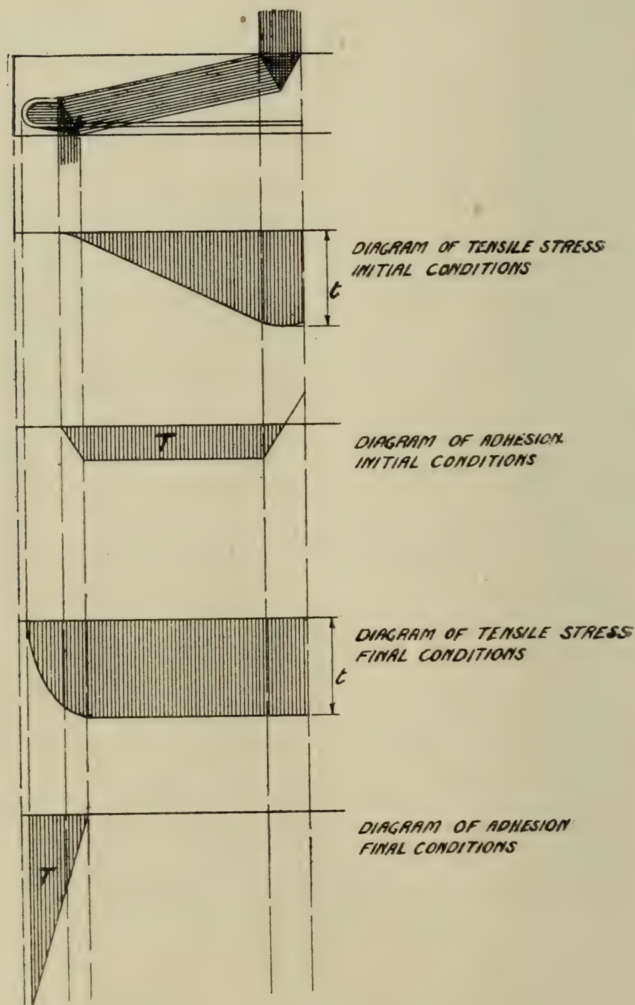


FIG 24

SHOWING THE VARIATION IN DISTRIBUTION OF TENSILE STRESS AND ADHESION STRESS WHEN A BEAM CHANGES FROM NORMAL WEB CONDITIONS TO CONDITIONS OF "DIRECT INCLINED COMPRESSION."

and proportional to their moduli respectively—a theorem which of course assumes no slipping, whereas we have already explained that we realize that the steel has slipped and relies on the friction at the support and the anchorage at the hook for its tension.

Those who still find any difficulty in imagining steel in tension adjacent to concrete in compression should consider the forces produced in a block of concrete through which is a steel bar, anchored at the top with a hook or washer and supporting a load at its lower end, as in Fig. 25.

A second difficulty may be urged in that the condition described cannot be attained until the web concrete has failed in diagonal tension, and that it is not desirable to use in practice beams in which cracks occur. The answer to this is that the cracks are so fine that at working loads they cannot, as a rule, be detected by the eye, and that they are unobjectionable since the tensile strength of the concrete is no longer depended on. It must be remembered, too, that exactly similar cracks are always formed in the concrete before the moment of resistance is attained, stressing the steel to 16,000 lbs., and no designer of experience will be found to argue that these latter cracks are dangerous.

It is interesting to note in passing that there is no difficulty in making the conditions described consistent with the usual assumption that plane sections shall remain plane after bending, provided the sections are taken at right angles to the resultant compression, though the writer would not consider it a serious criticism if this assumption had to be discarded. Karl Pearson has already shown for dams that engineers are not consistent in this matter, and that though they start off with this assumption in respect of certain planes under consideration, if the shearing and other resultant stresses are calculated, it will be found that the results of their calculations prove other planes to be unplane after bending.

We will now consider the maximum shear which can be resisted by such a system, from the point of view of its use in practical design.

The conditions to be satisfied are evidently as follows :—

(1) Steel must not be overstressed.
(2) Concrete must not be overstressed.
(3) No slipping must occur at the end.
(4) The system of internal forces must at every point equilibrate the external forces.
(5) Plane sections shall remain plane after bending.

As regards the first, no difficulty will be found in seeing that it is complied with.

As regards the third, an efficient hook or other fastening is obviously essential to the system. This will be considered later.

As regards the fourth, it should be noted that if the tension in the bottom member is constant from end to end, the horizontal component of the compression must also in pure beams be equal from end to end, since the total horizontal forces must balance.

If now we draw the line of the resultant compression within the material of the beam, and to such a scale and shape that the vertical distance between the centres of compression and tension represents the bending moment at that section (that is, make the shape of the compression a bending moment diagram to a certain scale), we have made the internal moments equate the external moments, and it will be found that the vertical component of the inclined compression exactly balances the external shear.

We have then completely complied with the fourth condition, and except near the hook at the ends there is no real shearing stress in the true sense on the concrete at all, but simply pure compression.

As regards the fifth, the planes will be taken at right angles to the principal stress, and the law is not necessarily complied with on all other planes.

The following analyses show the procedure when the safe shear is limited by the concrete stress in the web (the second condition).

7. Direct inclined compressions in rectangular beams.

It will be seen by reference to Fig. 26 that the maximum stress in the inclined compression will occur at the centre and at the end.

The stress at the centre will be safe if the beam has been designed for resistance moment in the usual way, the stress in that case varying from a maximum of C (not more than 600) at the upper face to zero at a depth of n.

If we consider the conditions of equilibrium of a vertical section just outside the support, and neglect the resistance to tension of the concrete at the top of the beam, it will be seen that the only forces acting are the horizontal com-

pression from the hook, and the tension in the bar. To maintain equilibrium, the resultant of the first must be on the centre of the bar (assuming the bars incapable of resisting appreciable bending stresses).

It follows that if d_1 is the distance of the bar from the lower surface, the stress at the end will vary from C_2 at the lower surface to zero at a distance of $3\,d_1$ up.

It follows, as the total compression is assumed constant from support to load, that if d_1 is at least equal to $\frac{n}{3}$, C_2 will not be greater than C_1, and will therefore be safe.

It will be found that when the horizontal component of the inclined compression can be equal to the flange forces of the resisting couple at the centre, the vertical component will necessarily be equal to the vertical shear.

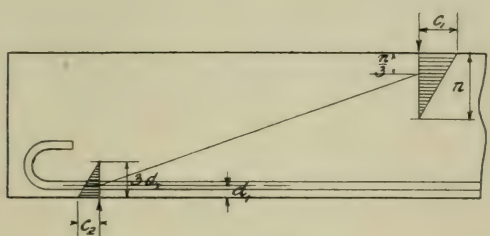


FIG 26

TO SHOW STRESSES IN A SIMPLE BEAM RESISTING THE WHOLE SHEAR BY INCLINED COMPRESSION

In other words, for a simple beam with a sufficient hook, with reinforcement running the whole length at a distance from the bottom of d *not less than* $\frac{n}{3}$ *failure will not occur until the full moment of resistance has been reached.*

This is a most important result, and shows that no shear reinforcement is ever necessary in such a beam, since, if the safe stresses are not exceeded by the bending moment at the centre, they will not be exceeded elsewhere.

It proves, also, what the writer has long maintained, that it is useless to attempt to test the safe resistance to shear of concrete beams unless the test beams are constructed as T beams, or heavily reinforced in compression.

It will be shown in the following pages that even if the bars are not quite $d_1 = \frac{n}{3}$ from the bottom, the safe stress will not necessarily be exceeded at the ends, and the above conclusions may still hold.

8. Approximate limit of shear resisted by direct inclined compression in non-continuous T beams, or rectangular beams with compression reinforcement.

In such beams the resistance to compression at the centre has been increased by the addition of the flange of the T or the compression bars, and as this increase of resistance does not help the web the limit to the inclined compression will often be found in the latter near the support.

To obtain an approximate idea of what the inclined compression is worth let us take

$$n = 0{\cdot}36\,d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

whence—

$$\frac{n}{3} = 0{\cdot}12\,d$$

$$d_1 = 0{\cdot}12\,d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

whence—

$$h = d + d_1 = 1{\cdot}12\,d$$

and—

$$a = 0{\cdot}88\,d$$

and—

$$d_1 = 0{\cdot}136\,a \quad \text{(see Fig. 27)}$$

These two assumptions (1) and (2) are generally near the truth in practical cases.

In this approximate analysis we will neglect the possible assistance of concrete or top steel at the end. (Considered later.)

Then the maximum horizontal compression of inclined compression that can be resisted—

$$H = 3\, d_1 \times d \times \frac{c}{2}$$

As the shear resistances are generally put in terms of b, a, it will be convenient to do the same and rewrite the above—

$$H = 0{\cdot}204\, a\, b\, c \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The rise of inclined compression is a.

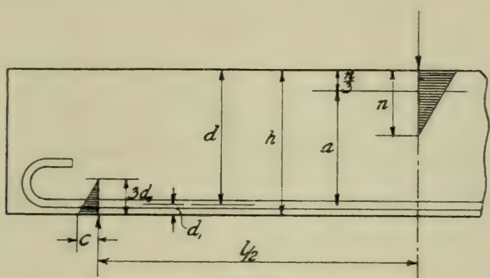


FIG 27

TO SHOW APPROXIMATE LIMIT OF SHEAR RESISTED BY INCLINED COMPRESSION

For single concentrated loads the rise is uniform in a distance $\frac{l}{2}$ (where l is the span).

Hence—

$$\text{Shear} = 0{\cdot}204\, a\, b\, c \times \frac{a}{\frac{l}{2}}$$

$$= 0{\cdot}408\, a\, b\, c \times \frac{a}{l}$$

Putting $c = 600$, this gives the following values for difference ratios of $\frac{l}{a}$:—

$$S = \frac{244\, b\, a^2}{l}$$

Span divided by Radius Arm.	Resistance to Shear.
$\frac{l}{a}=2$	$122\ b\,a$
4	61
5	49
10	24·5
15	16·2
20	12·2

It will be noticed that up to $\frac{l}{a}=4$ the resistance to shear exceeds the safe shear by diagonal tension if taken at $60\ b\,a$ as in L.C.C. Regulations.

For uniform loads.—Remembering that the slope of the parabolic stress curve will be a in a length $\frac{l}{4}$, we have as before—

$$\text{Shear} = 0{\cdot}204\ a\,b\,c \times \frac{a}{\frac{l}{4}}$$

$$= 0{\cdot}816\ a\,b\,c \times \frac{a}{l}$$

Span divided by Radius Arm.	Resistance to Shear.
$\frac{l}{a}=2$	$244\ b\,a$
4	122
5	98
10	49
15	32·4
20	24·4

It will be noticed that for $\frac{l}{a}$ up to 8, the safe shear exceeds that allowed by the L.C.C. under the formula 60 $b\,a$.

To make the formula applicable to all cases, let—

l_1 = the "point load span," that is, the span on which the load on the beam would produce the actual maximum bending moment if concentrated at the centre.

In any case l_1 may be calculated from the above, or graphically as in Fig. 37*a*.

It will be obvious that for—

Single concentrated loads ... $l_1 = l$

Uniform loads ... . $l_1 = \frac{l}{2}$

Two point loads at third points $l_1 = \frac{2}{3}l$

Then the formula may be written—

$$S = 244\frac{b\,a^2}{l_1}$$

or—

$$S = 150\frac{b\,h^2}{l_1}$$

It should be noted that not only are the results often greater than the shear obtained from diagonal tensions, but they may be used in combination with other systems for resisting shear, whereas diagonal tensions, for reasons given in (4), may not.

For rolling loads.—In the case of rolling loads the resistance to shear depends not only on the rates of span to depth, but also on the position of the load, and it so happens that the resistance is greatest where the shear is greatest (*i.e.* when the load is nearest the support), which makes this type of shear resistance specially valuable for such loads.

If the load W is at a distance of $x\,l$ from the left-hand support, the shear resistance by inclined compression is given by—

$$S = 0{\cdot}204\,a\,b\,c\frac{a}{x\,l}$$

and the shear across the section by—

$$S = \frac{W \times l(l-x)}{l} = W(1-x)$$

It will be seen that the first varies from infinity for $x\,l=0$, to something small when $x\,l=l$, whereas the second varies from $S=W$ when $x\,l=0$, and $S=0$ when $x\,l=l$ or $(x=1)$.

Although, therefore, the resistance is always adequate with the load in either extreme position, there may be conditions with the load near mid-span when the resistance by inclined compression is not sufficient acting alone.

9. Calculation of inclined compression taking tension of concrete into account in simple beams.

It will be shown in the following analysis that by taking the tensile resistance of the concrete into account, the value of the inclined compression may be increased.

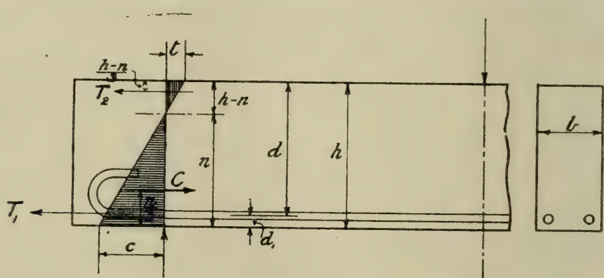


FIG 28

TO SHOW INCREASE OF INCLINED COMPRESSION IN A SIMPLE BEAM WHEN TENSION IN CONCRETE IS CONCRETE

In the previous analysis of the conditions at the support we found that the compressed area at the end had to be imited to a height of $3\,d_1$ from the lower surface so as to equilibriate the tension in the bars T_1.

It will be seen if a tensile force T_2 exists near the top surface, the point of action of C (see Fig. 28) need not coincide with T_1, but may be above it, and in that case C may have a considerably greater value.

It is thought that the conditions assumed in Fig. 28 are self-explanatory, with the remark that the bars T_1 are sup-

posed slipped, and hence free to have any stress apart from their position and distance from the neutral axis.

The section is assumed plane after bending.

We may at once write down the following equations—

$$\frac{t}{c}=\frac{h-n}{n} \quad . \quad . \quad . \quad . \quad . \quad (1$$

because stress diagram is linear.

Equating the total forces—

$$T_2+T_1=C$$

$$b\,(h-n)\frac{t}{2}+T_1=b\,n\frac{c}{2} \quad . \quad . \quad . \quad (2)$$

Equating moments about the bottom face—

$$T_2\left(h-\frac{h-n}{3}\right)+T_1\,d_1=C\,\frac{n}{3}$$

$$b\,(h-n)\,\frac{t}{2}\left(n-\frac{h-n}{3}\right)+T_1\,d_1=b\,n\frac{c}{2}\frac{n}{3} \quad . \quad (3)$$

Multiply (2) by d_1 and subtract from (3)—

$$b\,(h-n)\,\frac{t}{2}\left(h-\frac{h-3}{3}-d_1\right)=\frac{b\,n\,c}{2}\left(\frac{n}{3}-d_1\right)$$

Substitute $t=c\,\frac{h-n}{n}$ from (1)—

$$b\,(h-n)\,\frac{c}{2}\left(\frac{h-n}{n}\right)\left(h-\frac{h-n}{3}-d_1\right)=\frac{b\,n^2\,c}{2}\left(\frac{n}{3}-d_1\right)$$

Multiplying out and cancelling

$$n\,(-3\,h^2+6\,h\,d_1)+(2\,h^3-3\,h^2\,d)=0$$

$$n=\frac{h\,(3\,h-3\,d_1)}{3\,(h-2\,d_1)} \quad . \quad . \quad . \quad . \quad (4)$$

It is curious in cancelling out that the equation just misses being a cubic, the third and second powers all cancelling. This is due to the fact that the distance between T_1 and T_2 is a constant, and not a function of n as in other problems to follow.

It will be interesting to consider the special case when—

$$d_1 = 0{\cdot}12\, d \quad \text{(as before)}$$

$$= 0{\cdot}107\, h$$

$$n = \frac{(1 - 0{\cdot}321)\, h}{3\,(1 - 0{\cdot}214)} = 0{\cdot}712\, h$$

$$= 0{\cdot}91\, a$$

Note in passing how little n varies with d_1, being $0{\cdot}712\ h$ with $d_1 = 0{\cdot}12\ d$ and $0{\cdot}66\ h$ with $d_1 = 0$.

Having determined n, we can find what value of t corresponds to $c = 600$—

$$t = 600 \times 0{\cdot}282 = 235 \text{ lbs./in.}^2\ 0{\cdot}718, \text{ see formula } (1)$$

This is just about the ultimate value, and it would, therefore, be unsafe to make any use of the increase in the inclined compression due to it.

Note, however, that in **T** beams this stress would be much reduced, and in that case might be safe to use.

The above calculations would in any case not be often used as they give an inclined compression greater than the centre section is as a rule designed to resist, except in the case of beams with compression reinforcement near the centre, or beams deeper in the centre than at the end.

10. Calculation of inclined compression. Taking tension of concrete in account T beams.

In this case conditions are as before, except that at the top our tension flange has a width $B = M\, b$ (see Fig. 29) where M is a constant.

Our equations then become—

$$\frac{t}{c} = \frac{h - n}{n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$M\,b\,(h-n)\frac{t}{2}+T_1=b\,n\frac{c}{2} \quad . \quad . \quad . \quad . \quad (2)$$

$$M\,b\,(h-n)\frac{t}{2}\left(h-\frac{h-3}{3}\right)+T_1\,d_1=b\,n\frac{c}{2}\frac{n}{3} \quad (3)$$

Multiplying (2) by d_1 and subtracting from (3), then multiplying out and gathering terms, we get—

$$n^3\left(1-\frac{1}{M}\right)-n^2\times 3\,d_1\left(1-\frac{1}{M}\right)-n\times 3\,h\,(h-2\,d_1)$$
$$+h^2\,(2\,h-3\,d_1)=0 \quad . \quad . \quad (4)$$

This may be solved by trial for any special case, since n only varies quite slowly and regularly with M and d.

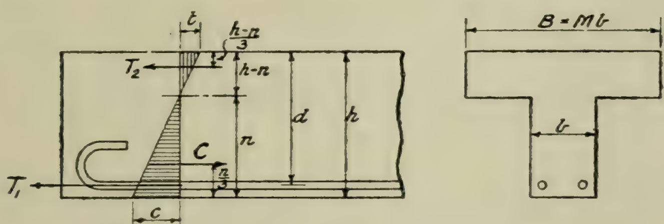


FIG 29

TO SHOW INCREASE OF INCLINED COMPRESSION IN T-BEAMS WHEN TENSION IN TOP FLANGE IS CONSIDERED.

To obtain some idea of the results this gives for practical cases, take—

$$d_1=0{\cdot}12\,d=0{\cdot}107\,h$$

and—

$$M=10 \text{ (i.e. } B=10\,b)$$

For this case, Equation (4) gives $n=0{\cdot}88\,h$—whence from (1) when $c=600$—

$$t=\frac{600\times 0{\cdot}12}{0{\cdot}88}=82 \text{ lbs./in.}^2$$

N 2

Having regard to the reduction of modulus when large stresses are reached, this is probably quite safe, and it should be noted it gives—

$$C = 0{\cdot}88\, b\, h\, \frac{c}{2}$$

instead of—

$$C = 0{\cdot}36\, b\, h\, \frac{c}{2}$$

when the tension in the concrete is neglected.

This is, of course, an important increase, and means that the safe shears in the tables on p. 190 are increased by about 50 per cent.

If we take—

$$d_1 = 112\, d$$

and—

$$M = 4$$

it will be found from Equation (4) that this gives—

$$n = 8{\cdot}25\, h$$

whence—

$$t = 0{\cdot}127 \text{ lbs./in.}^2$$

when $C = 600$.

This does not give a proper factor of safety, and the designer would have the alternative of halving both t and c (and thereby the resistance to shear by inclined compression) or of providing a little top steel and calculating as in the following analysis :—

11. Calculation of inclined compression taking tension in top steel into account.

For nomenclature see Fig. 30.

It must of course be remembered that the stress in the bottom steel t_1 is not determined by the strain diagram across the end section, since the bars are in tension, while the surrounding concrete is in compression.

The stress in the top steel, however, is so determined. Stress in concrete in tension is neglected.

Writing down the equations as before, we have—

$$t = 15\,c\,\frac{d-n}{n} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$A\,t + A_1\,t_1 = C \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$$A\,t\,d + A_1\,t_1\,d_1 = C\,\frac{n}{3} \quad . \quad . \quad . \quad (3)$$

$$C = b\,\frac{c}{2}\,n \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Multiply (2) by d_1 and subtract from (3).

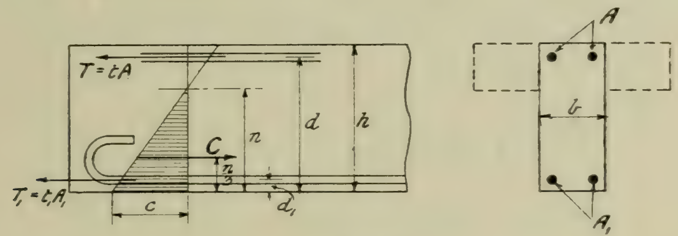


FIG 30

TO SHOW INCREASE OF INCLINED COMPRESSION WHEN TENSION IN TOP STEEL IS CONSIDERED

Substitute (1) and (4)—

$$A\,30\,C\left(\frac{d-n}{n}\right)(d-d_1) = b\,\frac{c}{2}\,n^2\left(\frac{n}{3} - d_1\right)$$

Whence—

$$n^3 - n^2\,3\,d_1 + n\,\frac{90\,A\,(d-d_1)}{b} - d\,\frac{90\,A\,(d-d_1)}{b} = 0$$

This is a cubic equation for n which it is frequently most convenient to solve by trial.

In the special case when—

$$A = 1 \text{ per cent.} = 0{\cdot}01\,b\,d$$
$$d_1 = 0{\cdot}12\,d$$

the equation gives—

$$n = 0{\cdot}74\,d \text{ or } 0{\cdot}84\,a \text{ (when } a = 0{\cdot}88\,d)$$

(Note this gives a stress of 3,160 lbs./in.^{2}.)
Then the inclined compression—

$$\begin{aligned} C &= 0{\cdot}84 \times 300 \times b \times a \\ &= 252\,b\,a \end{aligned}$$

Assuming the centre of compression at mid-span to be $0{\cdot}12d$ from the top. The rise is—

$$\left(0{\cdot}88 - \frac{0{\cdot}84}{3}\right)d = 0{\cdot}6\,d = 0{\cdot}682\,a$$

Shear resisted by inclined compression

$$= 252\,b\,a \times 0{\cdot}682 \times \frac{a}{\frac{L}{2}} \text{ for single point loads}$$

$$= 345\,b\,a\,\frac{a}{L} \text{ for single point loads} = 213\,\frac{b\,h^2}{L}$$

$$= 690\,b\,a\,\frac{a}{L} \text{ for uniform loads} = 426\,\frac{b\,h^2}{L}$$

The safe shear by inclined compression with 1 per cent. steel in top and $d_1 = 0{\cdot}12\,d$ is given in the following table.

Ratio of Span to Radius Arm.	Safe Shear.	
	Concentrated Loads.	Uniform Loads.
$\frac{l}{a}$		
2	172	344
3	69	138
10	34·5	69
13	23	46
20	17·2	34·5

It will be noticed that this gives results about 50 per cent. higher than when neither top steel nor top concrete are taken into account (see tables on p. 190).

(12) Value of n for maximum shear by direct inclined compression.

The shear resisted by inclined compression is—

$$S = C \tan \theta$$

where C is its horizontal component, θ is its inclination to the horizontal.

A little consideration will show that we can increase C by taking a large value for n, but by increasing n we bring

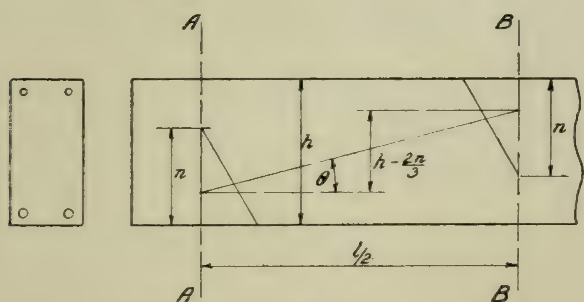


FIG 30a

the point of application of C nearer the centre of the beam, and thus reduce θ.

A point is reached when $C \tan \theta$ is a maximum, and beyond it an increase of n involves a reduction in the safe shear resistance.

Consider a rectangular beam, with the load applied at a section B B (Fig. 30a) and the support at the section A A, at a distance $\frac{l}{2}$ from it.

With a stress of 600 in the concrete at the lower fibre and zero at a distance n from it, the $C = b\,n\,\frac{600}{2}$.

If n is the same at sections A A and B B, as it generally

would be for rectangular sections (but not with T beams) we may put—

$$\tan\theta = \frac{h - \frac{2n}{3}}{\frac{l}{2}}$$

Hence the shear by inclined compression is—

$$S = b\,n\,\frac{600}{2}\,\frac{h - \frac{2n}{3}}{\frac{l}{2}} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

To obtain a maximum, we will equate the differential coefficient with regard to n to zero—

$$\frac{dS}{dn} = \frac{300\,b}{\frac{l}{2}}\left\{h - \frac{4n}{3}\right\} = 0$$

whence—

$$n = \frac{3h}{4}$$

This value gives the greatest possible resistance to shear for rectangular sections.

It is to be noted that this result agrees closely with the value obtained in section 11, where top steel is provided, and may with sufficient accuracy for most purposes be used in practice in such cases.

Substituting this value in (1) we get—

$$S = b \,.\, \tfrac{3}{4}h \,.\, 300 \,.\, \frac{h - \frac{h}{2}}{\frac{l}{2}}$$

$$= 225\,\frac{b\,h^2}{l} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

This is a simple and convenient formula for practical use where steel is provided top and bottom.

Where the load is not a central point load the formula may be written—

$$S=\frac{225\,b\,h^2}{l_1}$$

where l_1 is the "point-load span," as explained.

Note that with $n=0{\cdot}75\,h$, we have—

$$C=0{\cdot}75\times b\times h\times 300 \text{ lbs.}$$

This acts at a distance of $0{\cdot}25\,h$ from the bottom, or $0{\cdot}25-0{\cdot}107=0{\cdot}143\,h$ from the bottom steel, and $0{\cdot}75-0{\cdot}107=0{\cdot}643\,h$ from the top steel.

Hence tension required in bottom steel—

$$T=C\times\frac{0{\cdot}643}{0{\cdot}786}=225\,b\,h\times\frac{0{\cdot}643}{0{\cdot}786}=184\,b\,h$$

Limiting the steel to 16,000, the area of steel required is—

$$A=\frac{184\,b\,h}{16{,}000}=0{\cdot}0115\,b\,h, \text{ or say } 1{\cdot}2 \text{ per cent.}$$

13. **Direct inclined compressions in continuous beams.**

Consider a continuous beam with central point loads, as in Fig. 31.

The bending moment diagram will be as in (*c*), giving $\frac{Wl}{8}$ at centre and support, though it would be necessary to design for greater moments than these in cases where the simultaneous action of loads in all spans could not be relied on.

In such a beam, the top fibres would be in tension from support to $l/4$ each way, and the bottom fibres through the central half-span.

In practice a beam would generally be reinforced as in Fig. 31 (*a*), that is, the bottom bars would be carried through to the supports, while in the top reinforcement is supplied as far as the points of contraflexure only. (No doubt stirrups and bent bars would be used as well, but these need not be considered now.)

Consider now the forces acting across the action A A at the point of contraflexure.

As the moment at this point must be zero, and as there

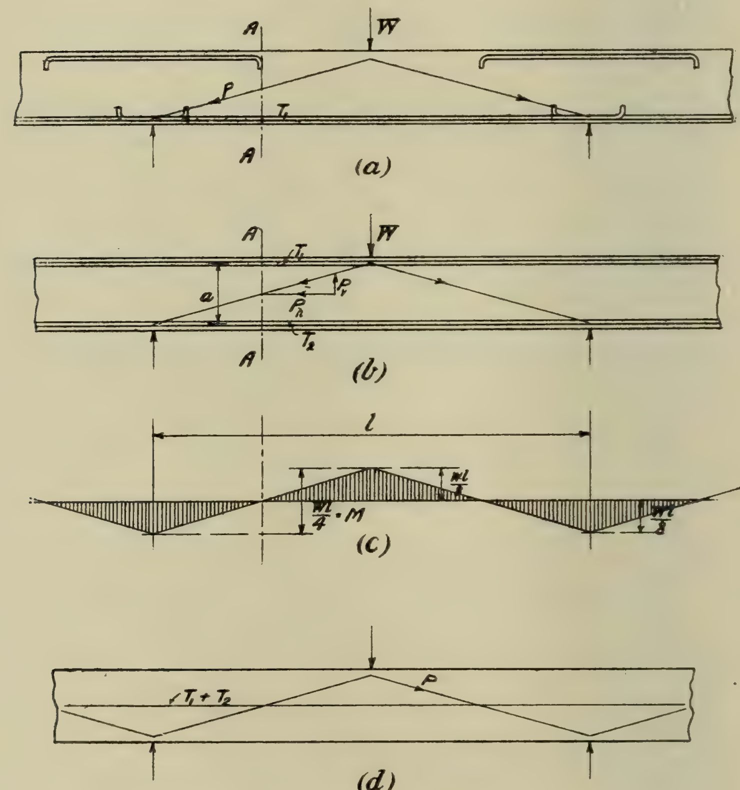


FIG 31

INCLINED COMPRESSION IN CONTINUOUS RECTANGULAR BEAMS WITH CONCENTRATED LOAD

can be no tension at the top (since the steel is stopped short), it follows that either—

(1) P and T_1 intersect here, or
(2) P is zero, or
(3) T_1 is zero

The first of these alternatives evidently does not apply. The second implies no shear resisted by inclined compression.

As regards the third, it will be seen that if we insist on true beam action (total tension equal to total compression) it implies P = zero, and therefore the same result, that no shear can be resisted by inclined compression.

If we do not insist on this condition, we are dealing with a line of continuous arches, not beams, and we will here confine ourselves to the latter.

We therefore have the following important result :—

In continuous beams, where the top reinforcement does not extend beyond the point of contraflexure, no shear may be resisted by direct inclined compressions.

This does not prevent very important resistance to shear by indirect inclined compressions. This is dealt with later.

Consider now Fig. 31 (*b*), showing a beam under similar conditions, but having the steel in top and bottom extended continuously through the various spans (though not necessarily without joints, provided they are well lapped and hooked or otherwise connected).

Consider now the conditions at the point of contraflexure in such a beam, where the whole shear is to be carried by inclined compression.

Since total tension = total compression we have—

$$P_h = T_1 + T_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Since the whole shear is to be taken by the inclined compression—

$$P_v = \frac{W}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Now—

$$\frac{P_v}{P_h} = \frac{a}{\frac{l}{2}} \text{ from the figure}$$

Substituting (1) and (2)—

$$2\left(\frac{W}{T_1 + T_2}\right) = \frac{a}{\frac{l}{2}}$$

whence—

$$T_1 + T_2 = \frac{W\,l}{4} \times \frac{1}{a}$$

Now $\frac{Wl}{4}$ is the M_t the total bending moment in the beam considering the ends free.*

Hence we have the following important result.

If in a continuous beam the whole *shear is to be taken by the inclined compression, continuous steel must be provided to take and carry a total tension equal to the total free bending moment divided by the radius arm.*

The horizontal component of the inclined compression must equal the same figure.

This means that the steel and concrete have to be just as ample as if the beam were designed as a free beam, the only change being that the steel is now partly along the top and partly along the bottom.

It would also be quite permissible to put the whole steel in the centre of the beam, as in 31(*d*), when it will be seen that the internal moments still equal the external moments at every point.

A careful consideration of the problem will show that the formula—

$$S=225\frac{bh^2}{l}$$

already derived in section 11*a* applies to continuous beams, the only difference being that the steel required—

$$A=0{\cdot}014\,bh\left(=\frac{225\,bh}{16{,}000}\right)$$

must be distributed between the two flanges, instead of being concentrated in the lower one.

The steel required in the bottom flange is—

$$0{\cdot}014\,bh\times\frac{M_c}{M_t}$$

and in the top flange is—

$$0{\cdot}014\,bh\times\frac{M_s}{M_t}$$

* Note that $M_t = M_c + M_s$ in symmetrical arrangements, where M_c is the centre moment and M_s is the moment at the supports. In this case $M^t = \frac{Wl}{4}$ and $M_c = M_s = \frac{Wl}{8}$.

In the cases where, owing to different loadings of adjacent panels, M_c and M_s may vary, the worst case must be taken. In that case the total steel required may exceed 0·014 B h.

Thus with a uniform load—

$$M_t = \frac{W\,l}{8}$$

and if we require to design centre and support for—

$$M_c = \frac{W\,l}{12} \quad \text{and} \quad M_s = \frac{W\,l}{12}$$

we have area of steel required in top—

$$= 0{\cdot}014\, b\, h \times \frac{8}{12}$$

and the same in the bottom.

This gives a total area of $0{\cdot}014\, b\, h \times \frac{16}{12} = 0{\cdot}0186\, b\, h$.

In the case of a beam where so much steel is not provided, we may use the formula—

$$S = 150\, \frac{b\, h^2}{l_1}$$

in which case the top steel required is—

$$0{\cdot}006\, b\, h \times \frac{M_s}{M_t}$$

and the bottom steel required is—

$$0{\cdot}006\, b\, h \times \frac{M_c}{M_t}$$

as a practical example of a continuous beam with considerable top and bottom steel carried right through, consider a silo wall resisting pressure from either side alternately.

Here we have—

$$M_t = \frac{W\,l}{8}$$

Suppose now we design the ends and centre for—

$$M_c = M_s = \frac{W\,l}{12}$$

and design the slab by the formula—

$$M_c = M_s = \frac{W\,l}{12} = 95\,b\,d^2 \quad . \quad . \quad . \quad (1)$$

Using 0·675 per cent. of steel on each side measured on $b\,d$.

If the distance from the C.L. of steel to the edge of the slab is $0{\cdot}12\,d$, as it will generally be approximately, we may put $h = 1{\cdot}12\,d$.

$95\,b\,d^2$ may be written $0{\cdot}76\,b\,h^2$ and $0{\cdot}00675\,b\,d$ may be written $0{\cdot}006\;b\,h$.

We therefore have, from (1)—

$$\frac{W\,l}{12} = 76\,b\,h^2 \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Notice, now, that the area of steel required in each flange to justify—

$$S = 225\,\frac{b\,h^2}{l_1} \text{ is } 0{\cdot}014 \times \frac{8}{12} = 0{\cdot}0093\;b\,h$$

whereas to justify—

$$S = 150\,\frac{b\,h^2}{l_1} \text{ is } 0{\cdot}006 \times \frac{8}{12} = 0{\cdot}04\;b\,h$$

As we are providing 0·006, which lies between these values, we may take—

$$S = \left\{150 + 75 \times \frac{0{\cdot}2}{0{\cdot}53}\right\}\frac{b\,h^2}{l_1} \text{ or } 178\,\frac{b\,h^2}{l_1}$$

Substituting $l_1 = \frac{l}{2}$ for uniform loads—

$$b\,h^2 = \frac{W\,l}{12 \times 76} \text{ from (2)}$$

We have safe shear—

$$S = 172 \times 2\,\frac{W\,l}{12 \times 76} = 0{\cdot}39\,W\,l$$

The actual shear is, of course, $0{\cdot}5\,W\,l$, so that if designed on the basis above mentioned, $0{\cdot}78$, or rather over $\frac{3}{4}$, of the total shear can safely be resisted by inclined compression. If the *whole* shear were resisted in this manner, it only means that the concrete will be slightly overstressed to—

$$600 \times \frac{0{\cdot}5}{0{\cdot}39} = 770 \text{ lbs./in.}^2$$

14. Indirect inclined compressions.

It has been shown that in continuous beams the value of *direct* inclined compressions in resisting shear is frequently small. This, however, does not prevent the value of *indirect* inclined compressions being great. Consider the arrangement of bars in Fig. 32 (*a*) which is a very common one.

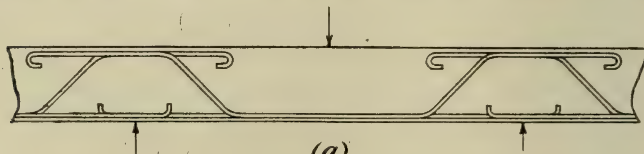
For purposes of analysis we may divide this up into systems 1 and 2 as shown in Figs. 32 (*b*) and (*c*), and fill in the inclined compressions shown, which will enable the reinforcement to act as a truss.

A careful consideration of these inclined compressions teaches several lessons in the design of reinforced concrete members, which are difficult to realize otherwise. Notice first that system (2) is complete in itself, and the stress in each member is easily determinable by the method of sections or by a funicular polygon.

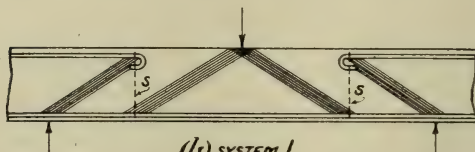
Note then that system (1) is only complete if the stirrup S is provided, and made adequate. This is a very important result, because it shows that stirrups are an integral part of any system in which the bars, without being taken from one flange to the other, are stopped off between the support and the load.

Yet the arrangement is sometimes used without stirrups,

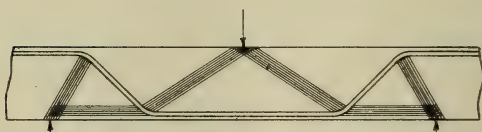
showing that some designers do not appreciate this point. They probably hardly realize that they are relying on the tension of the concrete, since they use the system in com-



(a)

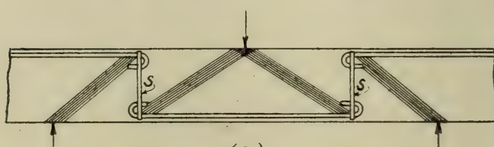


(b) SYSTEM 1

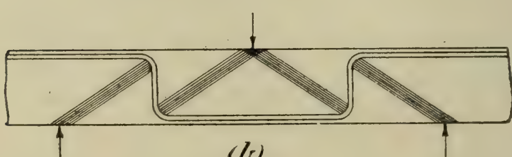


(c) SYSTEM 2

FIG 32 INDIRECT INCLINED COMPRESSIONS



(a)



(b)

FIG 33

INDIRECT INCLINED COMPRESSIONS

bination with a system of bent-up bars, in which a stress of 16,000 in the steel must necessarily mean air cracks in the concrete, after which its value in tension is of course nil.

The same point is illustrated in Fig. 33 (a), where the stirrup is an integral part of the system—except in the case of slabs where the diagonal tension in the concrete can take the *whole* shear.

Note that a bent-up bar as in Fig. 33 (b) has the same effect, and is in the writer's opinion much better, since there is less likelihood of the bar being seriously displaced when concreting. Note also that the stirrup also serves to resist the tendency of the hooks to open.

A system such as in Fig. 33 should only, as a rule, be used in combination with other systems, since it implies a certain position for the point of contraflexure, whereas this is generally subject to some uncertainty, and to variation with different loadings.

Note also that if the bent-up bar in Fig. 33 (b) is inclined as in Fig. 32 (c), this latter difficulty is partly removed, since with this system the point of contraflexure may vary between the points a and b without causing the system to be incomplete.

15. Maximum shear in a beam.

It was shown in sections 2, 3, and 4 how to calculate the resistance to shear of beams reinforced with stirrups and bent-up bars.

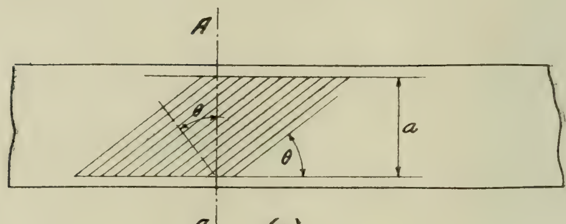
It was assumed, however, that failure would take place in the steel, whereas it is obvious that if the area of bent-up bars or of stirrups is increased sufficiently, failure will take place by the compressive stress, caused by these members on the concrete. Hence it is necessary to determine a limiting value to the safe shear in a beam of a certain size, *however heavily reinforced in shear*, and it is the object of this section to attempt this. If we consider any vertical section, resistance to shear will be due to inclined compressions crossing this section—whether direct, due to stirrups, or due to bent-up bars, matters not—or to inclined tensions crossing it.

Considering the inclined compressions first, let them be inclined at an angle θ to the horizontal (see Fig. 34 (a)) in a beam in which the radius arm of the flange force is a.

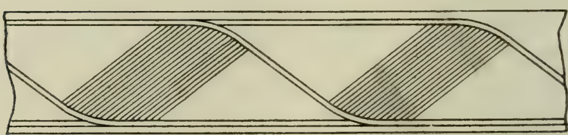
Then the width of the belt of the inclined compressions which cross any particular vertical section is—

$$a \cos \theta$$

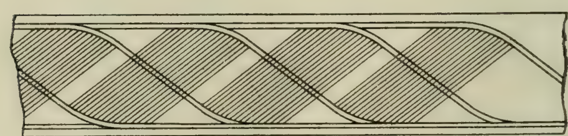
Hence, taking b as the breadth of the beam, and c the per-



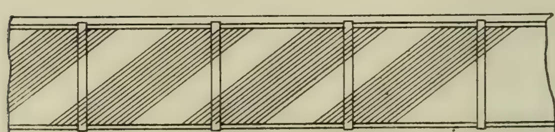
(a)



(b) SINGLE SYSTEM OF INCLINED BARS



(c) DOUBLE SYSTEM OF INCLINED BARS



(d) STIRRUPS

FIG 34
TO ILLUSTRATE LIMITING VALUES OF SHEAR

missible stress in the concrete, we have the limiting value of the inclined compression crossing the plane as—

$$C = a\,b\,c \cos\theta$$

and its vertical component available for resisting shear is—

$$S = a\,b\,c \cos\theta \sin\theta \quad . \quad . \quad . \quad . \quad (1$$

This is a maximum when $\theta = 45°$, where $\cos\theta \sin\theta = \frac{1}{2}$. Hence we may write—

$$S_{max.} = \frac{a\,b\,c}{2}$$

or putting—

$$c = 600$$

$$S_{max.} = 300\,a\,b$$

This is an extremely important result, and shows that across any vertical plane devoid of inclined bars, the shear may never exceed 300 *a b.*

Consider now the bent-up bar in 34 (*b*).

If its resistance is not limited by its own strength, it will be limited by the inclined compressions in the concrete in the adjacent panels, which clearly have to resist the same shear as the bar.

Hence we have the important result that—

The resistance of bent-up bars must never exceed $a\,b\,c \cos\theta \sin\theta$, *or* 300 *a b if* θ *is* 45° *and c is* 600.

Consider now the difference between the arrangement in 34 (*b*) and (*c*).

In the former the shear is carried by inclined compressions in one panel, inclined tension in the next, and so on alternately. This system is characterized by a bent bar every other panel, and is known as a single system of bent-up bars. Since the steel and the concrete each have to carry the whole shear, we see that

In a single system of bent-up bars the resistance is limited to 300 *a b, or less where* θ *is not* 45°.

In 34 (*c*) any vertical section such as A A is cut by inclined compressions and tensions, and as each may have a resistance equal to 300 *a b*, we have that

In a double system of bent-up bars the resistance is limited to 600 *a b or less when* θ *is not* 45°.

Since the whole resistance of stirrups is due to inclined compression, we have

resistance of stirrups is limited to $300\, a\, b$.

Note that if a system of stirrups, as in 34 (*d*), is superimposed on a single system as in (*b*) the inclined compression will be superimposed, and the limiting value is not increased.

In a system carried partly by stirrups and partly by a single system of bent-up bars, the resistance may never exceed $300\, a\, b$.

The effect of a superimposed stirrup system on a *double* bent-up system is to reduce the proportion of shear carried by the bent-up bars, and it may be shown that in a system carried partly by stirrups and partly by a double system, the resistance may never exceed

$$300\, a\, b \left(1 + \frac{S_b}{S_s + S_b}\right)$$

where S_b is shear taken by bent-up bar system
S_s ,, ,, stirrup system

It will be seen that this gives $300\, a\, b$ when stirrups take the whole, and $600\, a\, b$ when the double system takes the whole.

In the foregoing it has been assumed that the compression stresses were uniformly distributed across the width and breadth of the belts in which they act, and it remains to consider how far this is the case.

In Fig. 35 (*a*) are shown bent bars gently bent, in which the compression is well distributed across the panel, while in 35 (*b*) the bend is so sharp that this cannot be the case.

It is still not realized by many experts that (*b*) is bad and that (*a*) is essential if higher values of $\frac{S}{a\,b}$ are to be safely resisted. Where the bends are made as (*b*), the foregoing safe values for $\frac{S}{a\,b}$ must be considerably reduced.

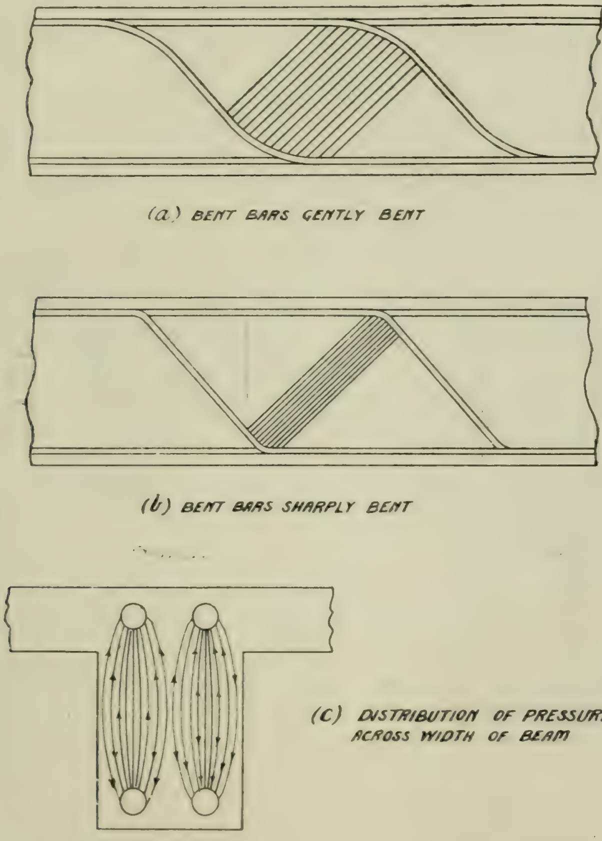
Consider now 35 (*c*), a section through beam showing the distribution laterally. It is obvious that though the pressure may be approximately uniform half-way down the web, it will be much more intense at the surface of the bars.

This applies equally whether stirrups or bent-up bars are under consideration.

It may be thought that the effect of this would be that the effective width of the beam should be reduced to the sum of the diameters of the bars and the stress taken on this area

only, but both experience and theory indicate that it is not necessary to go as far as this.

Consider Fig. 36.—When the pressure is uniform on the whole width as at (*a*), failure will occur along a plane inclined



(a) BENT BARS GENTLY BENT

(b) BENT BARS SHARPLY BENT

(C) DISTRIBUTION OF PRESSURE ACROSS WIDTH OF BEAM

FIG. 35

DISTRIBUTION OF INCLINED COMPRESSION

at an angle θ and the resistance will be proportional to the length $c\,d$, which is—

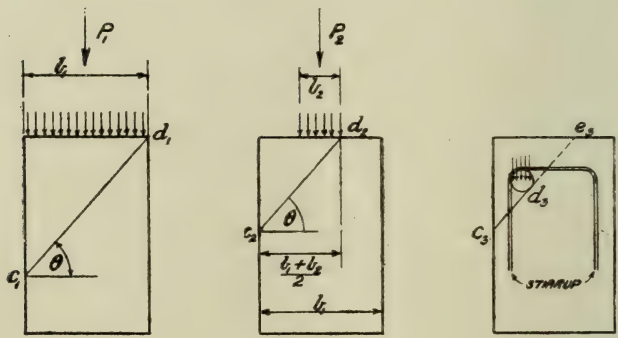
$$\frac{b_1}{\cos\ \theta} = P_1$$

If the pressure is concentrated on a small central area of width b_2 the resistance will be proportional to the length C_2 d_2, which is—

$$\frac{b_1 + b_2}{2 \cos \theta} = P_2$$

Hence—

$$\frac{P_1}{P} = \frac{b_2}{\frac{b_1 + b_2}{2}}$$



(a) (b) (c)

FIG 36

TO SHOW THE EFFECT OF LOCAL PRESSURE

In other words the concrete in case (*b*) behaves as if the width is neither *b* or not *b* but the mean of the two—

$$\frac{b_1 + b_2}{2}$$

Consider now 36 (*c*) showing the bar acting, as in practice, within the concrete. The effect is that not only has failure to occur along the plane c_3 d_3 as before, but the resistance of the portion d_3 e_3 or some other plane is called into play also, which has the effect of adding further to the safe pressure. If the stirrup as indicated on the drawing is provided, it will further prevent failure across the plane indicated.

Taking all these factors into account, we see that a much

higher local stress on the bars than 600 is perfectly permissible. The writer is of opinion that where the sum of the diameters of bars is equal to at least one-third the width of the web, a stress of 600 across the web is permissible where stirrups going across from one bar to the other laterally are provided.

If the sum of the diameters is less than one-third of the width of the web, the writer would take three times the sum of diameters of the bars as the effective width of the web.

Of course only the bars side by side are considered, and not those above one another.

This is equivalent to allowing a local pressure on the bars of 1,800 lbs./in.^{2}.

16. Two inclined compressions superimposed.

It sometimes happens that in a beam two inclined compressions of different slope may be superimposed. For example, a *direct* inclined compression from support to load may cross an *indirect* inclined compression produced by stirrups or bent-up bars. The latter will always be more inclined to the vertical.

The treatment of such cases is as follows :—

Where stress in the concrete in the web is not the limiting factor, the shear resistance due to the two may be added in full and no trouble arises.

Where, however, concrete stress in the web *is* the limiting factor, take the shear due to the steeper compression in full. If this stresses the concrete up to 600 the resistance of the lesser inclined must be neglected. If, however, it does not stress fully up to 600, calculate how great a stress may be permitted along the less inclined direction to give 600 as the resultant of the two, and allow the resistance to shear of this portion only, added to that of the more inclined compression.

Thus in Fig. 37, a steep compression "A" crosses a less inclined compression "B."

To calculate the resistance of this section of the beam to shear, calculate the stress a due to "A" and allow this in full up to a limit of 600.

If it is less than 600, set it down to a scale as shown, and with one end as centre draw an arc of a circle having 600 as radii. Then from the other end draw b parallel to B till it meets the arc, which will give the permissible stress in B which will not overstress the concrete.

This method of giving preference to the steeper compression gives the maximum resistance.

An example is given on page 227.

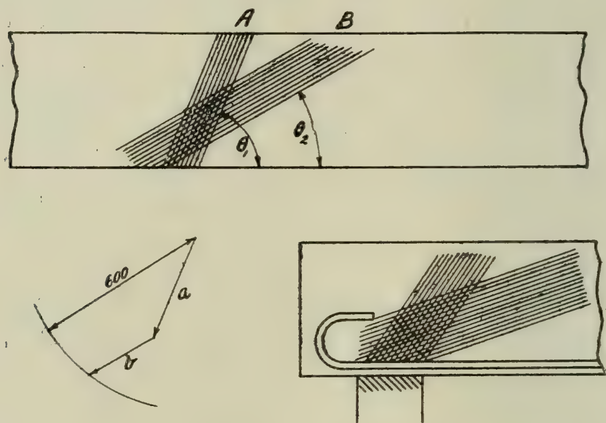
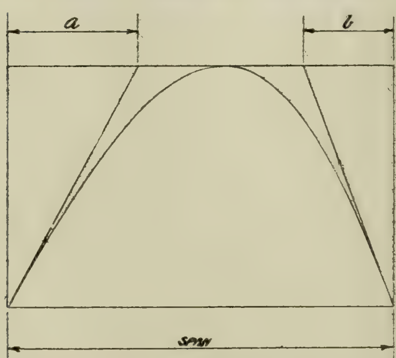


FIG 37
TWO INCLINED COMPRESSIONS SUPERIMPOSED



$l = a + b$

FIG 37 a
TO SHOW HOW TO FIND l FROM THE DIAGRAM OF MOMENTS (SEE SECTION 17)

It may be mentioned that where the lozenge 1, 2, 3, 4 (see Fig. 37*b*) subjected to the two stresses occurs near the support only, a combined stress of more than 600 is sometimes per-

missible, since the support gives lateral support and prevents lateral bursting.

Note that if instead of adding the compression stresses vectorially, as above, we add them arithmetically, the calculations are greatly simplified and the error is on the side of safety. The error is small where the inclinations vary by less than 45°.

17. Résumé of calculations of inclined compressions as affecting beams, with simple rules and formulæ.

It will be convenient here to gather together a few of the results now obtained in the preceding articles.

Where steel is provided on tension side only $n = 3\, d_1$ and

$$S = 1{,}800\, a\, b\, d_1 \quad . \quad . \quad . \quad . \quad (1)$$

if—

$$c = 600$$

when, further—

$$a = 0{\cdot}88\, d$$
$$d_1 = 0{\cdot}12\, d$$

this reduces to—

$$S = \frac{190\, b\, d^2}{l_1} \quad \text{or} \quad \frac{150\, b\, h^2}{l_1} \quad . \quad . \quad . \quad . \quad (2)$$

To justify this value the steel in tension must have an area of not less than—

$$A = 0{\cdot}006\, b\, h = 0{\cdot}00675\, b\, d$$

or—

$$p = 0{\cdot}675 \text{ per cent.} \quad . \quad . \quad . \quad . \quad (3)$$

carried to the support and provided with a *fully* efficient hook or fastening (see sections 6-8).

Where steel is provided on both sides—

$$n = 0{\cdot}75\, h$$

$$S = \frac{225\, b\, h^2}{l_1} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

To justify this value, the steel in tension must have an area of not less than—

$$A = 0\cdot014\, b\, h \quad \text{or} \quad 0\cdot0155\, b\, d$$

or—

$$p = 1\cdot55 \text{ per cent.} \quad . \quad . \quad . \quad . \quad (5)$$

carried to the support and provided with a *fully* efficient hook or fastening (see section 2).

The above formulæ are applicable to free and continuous beams. If free the specified area of steel must all be on the lower side, assuming a downward load, when (2) is used, and if formula (4) is used $A = 0\cdot012\, b\, h$ on the lower side, and at least half as much on the opposite side in addition.

If continuous, the specified area of steel must be divided between the top and bottom in the same ratio as the support moment and centre moment are to each other.

Thus if we have—

$$M_s = \frac{W\, l}{12} \quad \text{and} \quad M_c = \frac{W\, l}{24}$$

$$\text{then top steel} = \tfrac{2}{3}\, A$$
$$\text{bottom steel} = \tfrac{1}{3}\, A$$

A being the area required by formulæ (3) or (5).

If—

$$M_c = \frac{W\, l}{12} \quad \text{and} \quad M_s = \frac{W\, l}{24}$$

$$\text{then top steel} = \tfrac{1}{3}\, A$$
$$\text{and bottom steel} = \tfrac{2}{3}\, A$$

If the same beam, *both* these conditions may apply under different loadings, both top and bottom steel must be equal, $\tfrac{2}{3}$ A.

If the requisite area of steel is not provided or carried right through (measure it at its *least* section, S must be reduced in proportion between what is provided and what required.

In the preceding formulæ l_1 is the sum of the two distances from the supports to the points where the tangent to the

bending moment diagrams at the ends cuts a horizontal line drawn through the point of maximum moment (take free moment in continuous beams). See Fig. 37a, $l_1 = a + b$.

For single concentrated load ... $l_1 = \text{span}$

For uniform load ... ... $l_1 = \dfrac{\text{span}}{2}$

For point-loads at the third points $l_1 = \frac{3}{2}$ span

As the line of inclined compression is the bending moment diagram drawn very flat to lie within the section, the values given for S apply at the end where the shear is greatest, and fall off as the shear falls off.

Thus, for single concentrated load S is constant from support to load.

For two symmetrical loads S is constant from support to first load and zero between them.

For uniform load, S falls off uniformly to zero at mid-span.

Where other inclined compressions intersect the direct inclined compression where the stress is high (i.e. at load or support) the value of S given by formula (2) or (4) must be reduced so that the stress of $c = 600$ is not exceeded (see par. 16).

With this proviso, the shear by direct inclined compression may be added to that by stirrup s or bent-up bars.

However heavily reinforced, the safe shear across a section can never exceed $S = 300\ b\ a$ where stirrups alone, or a single system of bent bars alone, take the whole load.

And can never exceed $S = 600\ b\ a$ where a double system of bent bars carry the whole load.

With a combination of stirrups and a single system of bent-up bars $S = 300\ b\ a$ can never be exceeded.

With a combination of stirrups and a double system of bent-up bars, a value between $S = 600\ b\ a$ and $S = 300\ b\ a$ can never be exceeded.

If θ is different from $45°$, or if a direct inclined compression takes part load, the above values have to be reduced.

Beams should not be designed for a section less than corresponds to $S = 200\ b\ a$ or more except by a scientific designer of experience.

Shear Tests on T Beams (1914 Series).

(a) *Descriptions of Experiments.*

A series of test beams was designed by the writer to be tested in shear, special care being taken to ensure that they

FIG. 38.

should fail in this and no other manner, and also that they should prove or disprove as clearly as possible the writer's theory and analysis of inclined compressions.

The series is illustrated in Figs. 38 and 39.

It will be seen that it consists of sixteen beams of **T** section, having a flange 12 in. wide, 4 in. thick, and a web 5 in. thick and 8 in. deep.

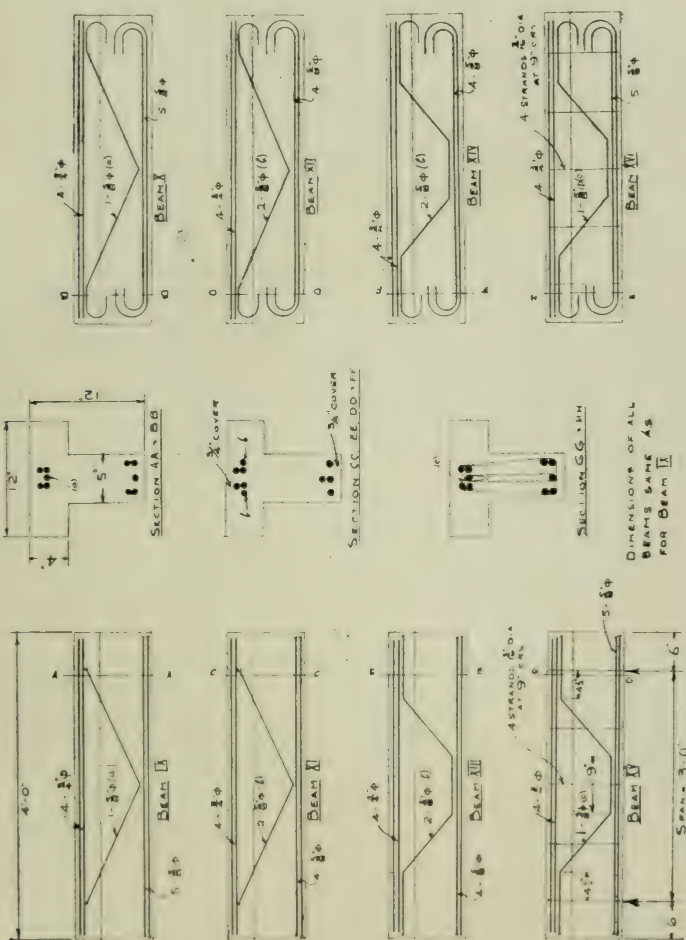


FIG 39

The tension reinforcement consists of six $\frac{5}{8}$-in. diameter bars in all beams, and the top flange is strengthened with four $\frac{3}{4}$-in. bars.

Beams 1, 3, 5, etc., are provided with fishtails at the ends

of the tension bars only, while 2, 4, 6, etc., have large hooks, beams 1 and 2, 3 and 4, etc., being otherwise identical.

The object of this was to make certain whether failure was due to slipping or to insufficient strength of hook or fishtail.

Beams 1 and 2 have no shear reinforcement, 3 and 4 stirrups, 5 and 6 more stirrups, and 7 and 8 still more.

Beams 9 and 10 have inclined bars, 11 and 12 more inclined bars, 13 and 14 inclined bars of greater slope, and 15 and 16 combinations of inclined bars and stirrups.

The steel was commercial mild steel having an ultimate strength 23·8 tons, and an elongation of 29 per cent. in 8 in.

The concrete consisted of—

4 parts	crushed flint	...	$\frac{3}{4}$ in. to $\frac{1}{4}$ in.
2 ,,	sand ...	...	$\frac{1}{4}$ in. to zero
1 part	Portland cement		

The latter gave an average of 700 lbs./in.² neat tension in seven days.

The beams were made by practical workmen engaged in the construction by the author's firm of Charterhouse Street Cold Storage for the Port of London Authority.

They were made :—

Beams	1–4	...	December 19, 1913
,,	5–8	...	December 23, 1913
,,	2–12	...	January 1, 1914
,,	13–16	...	January 14, 1914

and tested at the City and Guilds Engineering College in 1914, at an age of about four months.

The length of the beams was 4 ft., and they were tested by a central point load on a span of 3 ft.

The load was distributed at the supports over a width of 3 in., and at the centre over a width of 6 in. They were tested on a Riehle machine which was found very sensitive and suitable for the purpose.

(*b*) *Calculation of safe shear method of R.I.B.A. Report of* 1911.

	Safe Shear. lbs.	Safe Load. lbs.
Beams 1 *and* 2.		
Resistance of concrete to diagonal—		
tension = 60 $b\,a$	2,700	5,400

	Safe Shear. lbs.	Safe Load. lbs.
Beams 3 *and* 4.		
Resistance of concrete...	2,700	
2 strands $\frac{8}{16}$ in. at 9-in. centres—		
$2 \times 0{\cdot}0275 \times \frac{9}{9} \times 16{,}000$	880	
	3,580	7,160
Beams 5 *and* 6.		
Concrete	2,700	
4 strands $\frac{8}{16}$ in. at 9-in. centres ...	1,760	
	4,460	8,920
Beams 7 *and* 8.		
Concrete	2,700	
4 strands $\frac{8}{16}$ in. at $4\frac{1}{2}$-in. centres—		
$4 \times 0{\cdot}0270 \times \frac{9}{4\frac{1}{2}} \times 16{,}000$	3,520	
	6,220	12,440
Beams 9 *and* 10.		
Concrete	2,700	
1 $\frac{5}{8}$-in. bar slope of 1 in $2\frac{1}{4}$—		
$0{\cdot}3 \times \frac{4}{9} \times 16{,}000$	2,130	
	4,830	9,660
Beams 11 *and* 12.		
Concrete	2,700	
2 $\frac{5}{8}$-in. bars slope of 1 in $2\frac{1}{4}$—		
$2 \times 0{\cdot}3 \times \frac{4}{9} \times 16{,}000$	4,260	
	6,960	13,920
Beams 13 *and* 14.		
Concrete	2,700	
2 $\frac{5}{8}$-in. bars at 1 in $1{\cdot}42$—		
$2 \times 0{\cdot}3 \frac{16{,}000}{1{\cdot}42}$	6,780	
	9,480	18,960

	Safe Shear. lbs.	Safe Load. lbs.
Beams 15 *and* 16.		
Concrete	2,700	
1 $\frac{5}{8}$-in. bar at 1 in 1·42	3,400	
4 strands $\frac{3}{16}$ in. at 9-in. centres ...	1,760	
	7,860	15,720

(*c*) *Calculation of safe shear by L.C.C. Rules* (Reinforced Concrete Regulations, L.C.C. (General Powers) Act, 1909).

64.—The vertical shear taken by the concrete only shall be calculated on the compressed area of the web or on the web area for a depth equal to the arm of the resistance moment of the beam. The intensity of the shearing stress shall not be greater than the values given in Regulation 42.

65.—Where the vertical shear is taken by the concrete only, in accordance with Regulation 64, the ends of 50 per cent. of the bars of the tensile reinforcement shall be inclined across the neutral plane of the beam, and shall be carried through a depth equal to the arm of the resistance moment, or the whole of the bars shall be carried through to the ends of the beam (see Regulations 87 (*c*) and 88 (*c*)).

66.—If the shearing stress at any cross-section, calculated on the concrete alone, is in excess of the permissible shearing stress, the whole shear shall be provided for by the tensile resistance of the shear or web reinforcement acting in conjunction with the compressive stresses in the web, but in no case shall the ratio $\frac{S}{b_r d}$ exceed three times the shearing stress given in Regulation 42 (*a*), where—

b_r = mean *breadth* of the *rib* of a tee beam or the *breadth* of a *rectangular* beam ;
d = effective *depth* of the beam ;
S = total vertical *shearing* force at any cross-section.

(Also see Regulation 84.)

It will be seen that diagonal tension in concrete is not to be taken in combination with bent-up bars and stirrups (Regulation 66). So in beams 13 and 16 the resistance of bent-up bars and stirrups only is to be allowed. In beams 3 and 4 the stirrups give safe shear $= 2 \times 0·0275 \times 16,000 = 880$ lbs.

As, however, the concrete gives 2,700, and we have to take either but not both (see Regulation 66), the concrete value only is allowed.

A. Similarly in beams 3 and 4, 5 and 6. In 7 and 8 the stirrups give a higher result than the concrete, and this value is therefore taken.

In all subsequent beams the reinforcement gives higher results than the concrete and is therefore used in the calculations, except in the case of beams 9 and 10, where the steel gives only—

$$0 \cdot 3 \times \frac{4}{9} \times 16{,}000 = 2{,}130 \text{ lbs.}$$

and the concrete value of 60 $b\,a$ = 2,700 is therefore used.

Beam No.		Safe Shear.	Safe Load.
		lbs.	lbs.
1 and 2	60 $b\,a$	2,700	5,400
3 „ 4	60 $b\,a$	2,700	5,400
5 „ 6	60 $b\,a$	2,700	5,400
7 „ 8	4 strands at 4½-in. centres	3,520	7,040
9 „ 10	60 $b\,a$	2,700	5,400
11 „ 12	2 ⅝-in bars	4,260	9,520
13 „ 14	2 ⅝-in. bars, slope 1 in 1·42	6,780	13,560
15 „ 16	1 ⅝-in. bar 4 strands at 9-in. centres	3,400 1,760	10,320

(*d*) *Calculation of safe shear by the writer's analysis of inclined compression.*

Beams 1 *and* 2.—The beams are evidently subject to a most important direct inclined compression, and the first thing will be to calculate its amount.

Since we have top steel, we will use the analysis of section 11 (page 197).

This gives us an equation for n at the support.

$$n^3 - n^2 \, . \, 3\,d_1 + n\,\frac{90\,A\,(d - d_1)}{b} - d\,\frac{90\,A\,(d - d_1)}{b} = 0$$

Substituting—

$$d_1 = 1\tfrac{1}{2} \text{ in.}$$
$$d = 10\tfrac{1}{2} \text{ ins.}$$
$$A = 4\ \tfrac{3}{4}\text{-ins. bars} = 1{\cdot}76$$
$$b = 5 \text{ ins.}$$

we get—

$$n^3 - 4\tfrac{1}{2}\,n^2 + n\,\frac{90 \times 1{\cdot}76 \times 9}{5} - 10\tfrac{1}{2}\,\frac{90 \times 10{\cdot}76 \times 9}{5} = 0$$

It will be found that this is solved by—

$$n = 9{\cdot}15 \text{ ins.}$$

This gives the horizontal component of the inclined compression—

$$c = 9{\cdot}15 \times 5 \times 300 = 13{,}700 \text{ lbs.}$$

Note that causes a stress of $\frac{13{,}700}{6 \times 0{\cdot}3} = 7{,}600$ lbs.-in.² on the six bottom bars, which shows that they will not be overstressed, even in beams 11 to 14 when two bars are bent up.

Note also—

$$d - n = 10\tfrac{1}{2} - 9{\cdot}15 = 1{\cdot}35 \text{ in.}$$

Hence tension in top bars at support—

$$t = 600 \times 15 \times \frac{1{\cdot}35}{9{\cdot}35} = 1{,}300 \text{ lbs./in.}^2$$

$$T = 1{\cdot}76 \times 1{,}330 = 2{,}350 \text{ lbs.}$$

Note that this latter produces an adhesion stress in 5-in. length of 4 $\tfrac{3}{4}$-in. bars of—

$$\frac{2{,}350}{5 \times 4 \times \pi \times \tfrac{3}{4}} = 50 \text{ lbs./in.}^2$$

which is well within the allowable stress.

At the centre section we have $n = 4{\cdot}2$ in. (which it is not considered necessary to show here), and the centre of pressure $\frac{4{\cdot}2}{3} = 1{\cdot}4$ down from top (see Fig. 40).

Hence the rise of inclined compression is—

$$12 - \frac{9{\cdot}15}{3} - 1{\cdot}4 = 7{\cdot}55 \text{ ins.}$$

and occurs in a horizontal length of $15\frac{1}{2}$ in.

Hence the shear due to this is—

$$13{,}700 \times \frac{7{\cdot}55}{15\frac{1}{2}} = 6{,}670 \text{ lbs.*}$$

Coming now to *beams* 3 *and* 4, we have the case of a compression at 45° due to the stirrups superimposed on one at about 30° (direct).

Hence we have to follow the treatment given in section 16 (p. 215) and take the first in full, and so much of the second as will keep the resultant stress below 600.

The shear due to the stirrups is 880 as before, which produces a stress of—

$$\frac{880}{a\, b \cos\theta \sin\theta} = \frac{1{,}760}{9 \times 5} = 39 \text{ lbs./in.}^2$$

in the concrete.

From the construction given in the section referred to, it is found that to keep the resultant stress under 600, the stress in the inclined compression must be kept down to 565 (see Fig. 41).

Hence shear due to inclined compression is—

$$6{,}670 \times \frac{565}{600} = 6{,}280.$$

Similarly for *beams* 5 *and* 6 shear due to stirrups = 1,760 lbs.

This produces concrete stress = 78 lbs./in.^{2}.

By diagram, stress in direct inclined compression must be reduced to 530.

$$\text{Shear by inclined } \textit{compression} = 6{,}670 \times \frac{530}{600} = 5{,}900$$

* The simple formula $S = 225 \frac{b\, n^2}{2}$ (Part 3, secs. 11A and 17) gives $S = 5{,}200$. The formula was derived from rectangular beams, and a greater rise is obtained with T beams. Hence the difference.

Similarly for *beams* 7 *and* 8 shear due to stirrups = 3,520 lbs.

This produces concrete stress of 156 lbs./in.^{2}, direct inclined compression stress must be reduced to 453 shear by inclined compression $= 6{,}670 \times \frac{453}{600} = 5{,}030$.

The stress diagrams for all the above are on Fig. 41.

Coming now to *beam* 9, we notice that the inclined bar is in the condition mentioned in section 2 (p. 173) of being insufficiently fixed at its end, and if we consider the vertical section A A in Fig. 39 just clear of the support, we see that the stress is limited to the efficiency of the hook only. Taking this as one-half of the strength of the bar, we have—

Shear due to bent bar

$$= 0{\cdot}3 \times 16{,}000 \times \tfrac{1}{2} \times \tfrac{4}{9} = 1{,}065$$

Inclined compression ... $= 6{,}670$

In *beam* 10, the inclined bar is adequately fixed and may be taken in full.

Shear due to bent bar

$$= 0{\cdot}3 \times 16{,}000 \times \tfrac{4}{9} = 2{,}130$$

Inclined compression ... $= 6{,}670$

Similarly the inclined bars in *beam* 11 are badly fixed and give us—

Shear by inclined bars $= 2{,}130$

Inclined compression $= 6{,}670$

While in *beam* 12 they are fully fixed and give—

Inclined bars ... $= 4{,}260$

Inclined compression $= 6{,}670$

Note that in the last four beams the shear carried by the bent bars is taken up to the top flange over the support and is transmitted to it vertically. The vertical compression due to this might be treated as a superimposed compression, in

virtue of which the inclined compression should be reduced. This has not been done because—

(*a*) The forces are nearly at right angles.
(*b*) The overstressed part would be very near the support, where additional lateral support is given.

Coming now to *beams* 13 *and* 14, a steep inclined compression is produced from the top bend to the support, which we calculate in full and then reduce direct compression to keep within 600.

We have—

Resistance due to inclined bars

$$=2 \times 0{\cdot}3 \times 16{,}000 \times \frac{1}{1{\cdot}42}=6{,}780 \text{ lbs.}$$

Stress in concrete due to this is—

$$\frac{6{,}780}{a\,b \cos\theta \sin\theta}=\frac{6{,}780}{9 \times 5 \times \frac{1}{2{\cdot}24} \times \frac{2}{2{\cdot}24}}$$

$$=375 \text{ where } \theta=\tan^{-1}\frac{9}{4\frac{1}{2}}$$

Graphically in Fig. 42 we find this requires the direct compression to be reduced from 600 to 260 and gives—

Resistance due to inclined compression

$$=6{,}670 \times \frac{260}{600}=2{,}890$$

In *beams* 15 *and* 16 we take in full the resistance of stirrups and bent-up bar, calculate the compression due to these, and by means of the stress diagram find the permissible stress in the direct inclined compression. This is just as before, except that this time we have three forces in our vector diagram.

Resistance due to stirrup $=1{,}760$

Concrete stress due to this $=78$ at 45°

Resistance due to inclined bar $=3{,}390$

Concrete stress due to this $=187\ \theta=\tan^{-1}\frac{9}{4\frac{1}{2}}$

Hence by diagram in Fig. 43, stress in direct inclined compression is 365.

Resistance due to inclined compression

$$=6{,}670\times\frac{365}{600}=4{,}060$$

(*e*) *Results of experiments.*

The records of cracks and the loads at which they occurred are given in Figs. 44–47.

Photographs of the beams are given in Figs. 48–63.

It will be interesting first to take the ultimate loads from Figs. 44–47 and compare these with those calculated by R.I.B.A. methods and find the factor of safety by dividing the first by the second. This is done in the following table :—

Beam No.	Safe Load, R.I.B.A.	Ultimate Load.	Factor of Safety.
	lbs.	lbs.	
1	5,400	42,400	7·85
2	,,	47,600	8·80
3	7,160	54,850	7·64
4	,,	50,000	7·0
5	8,920	50,000	5·6
6	,,	48,500	5·42
7	12,440	56,800	4·54
8	,,	52,600	4·21
9	9,660	41,150	4·30
10	,,	56,800	5·88
11	13,920	57,800	4·15
12	,,	66,500	4·78
13	18,960	54,800	2·89
14	,,	55,680	2·93
15	15,720	63,070	4·02
16	,,	64,500	4·12

Comparing now L.C.C. safe loads with observed ultimate loads in the same way, we have—

Beam No.	Safe Load, L.C.C.	Ultimate Load.	Factor of Safety.
	lbs.	lbs.	
1	5,400	42,400	7·85
2	"	47,600	8·80
3	5,400	54,850	10·2
4	"	50,000	9·27
5	5,400	50,000	9·27
6	"	48,500	8·99
7	7,040	56,800	8·1
8	"	52,600	7·5
9	5,400	41,150	7·63
10	"	56,800	10·5
11	8,520	57,800	6·8
12	"	66,500	7·8
13	13,560	54,800	4·05
14	"	55,680	4·1
15	10,320	63,070	6·1
16	"	64,500	6·3

We will now make the same comparison with the safe load as calculated by the writer's method, and the ultimate load—

Beam No.	Safe Shear.	Safe Load.	Ultimate Load.	Factor of Safety.
	lbs.	lbs.	lbs.	
1	6,670	13,340	42,400	3·2
2	"	"	47,600	3·6
3	7,160	14,320	54,850	3·8
4	"	"	50,000	3·5
5	7,600	15,320	50,000	3·3
6	"	"	48,500	3·2
7	8,500	17,000	56,800	3·3
8	"	"	52,600	3·1
9	7,735	15,470	(41,150)	(2·7) *
10	"	17,600	56,800	3·2
11	8,800	17,600	57,800	3·3
12	10,930	21,860	66,500	3·1
13	9,670	19,340	54,800	2·8
14	"	"	55,680	2·9
15	9,210	18,420	63,070	3·4
16	"	"	64,500	3·5

* The experimental notes state that the ultimate load for beam 9 was not obtained, but was known to be greater than 41,150. Hence no value attaches to the figures in brackets.

It will be seen that the R.I.B.A. method gives factors varying from 8·8 to 2·9.

The L.C.C. methods gives factors varying from 10·5 to 4.

The writer's method gives factors varying from 3·8 to 2·8.

It would be seen that the series of experiments bears out in a remarkable manner the inclined compression theory as outlined in Part III of this thesis, having regard to the greater variation in the size and arrangement of shear members in the series, and to the necessary variation of hand-made concrete made at different dates by practical workmen.

About the R.I.B.A. results, it is curious that they give results as concordant as they do. It will be noticed that they ascribe a large value in all the beams to diagonal tension, although the fact that diagonal cracks occurred long before failure even in beams 1 and 2 proves that this had been destroyed, as theory indicates.

It must, therefore, be admitted that the concordance of the R.I.B.A. results is due to the fact that in this series the value they ascribe to diagonal tension is not so very different to the actual value of the inclined compression which actually existed.

It must be noted, however, that in long beams they would still ascribe to it the same value even in combination with shear reinforcement, although the value of the inclined compression would then be very much smaller, and in such cases the R.I.B.A. method of design would lead to dangerous results by allowing for a large diagonal tension in circumstances when it could not possibly exist.

The results show clearly how wasteful the L.C.C. rules are, which one is tempted to ascribe to the well-known attitude of the Local Government Board to reinforced concrete. Factors of safety of 8 are luxuries that we can ill afford in these times, or, indeed, in any times.

It is interesting now to compare the loads on the beams with fishtails with those with hooks as follows :—

Fishtails.	Hooks.
42,400	47,600
54,850	50,000
50,000	48,500
56,800	52,600
41,150	56,800
57,800	66,500
54,800	55,680
63,070	64,500
420,870	442,180

This agreement indicates clearly that failure did not occur by insufficient fixing at the support, since it would then have been too curious a coincidence that the fishtails should have the same value as the hooks.

As little information about the value of hooks and fishtails exists, it is interesting to note that the horizontal component of the inclined compression (calculated) was 13,700, and must have equalled $13{,}700 \times 3{\cdot}6 = 49{,}500$ lbs. in beam 2, since beam 2 gave a true factor of 3·6.

Subtracting from this the friction due to the vertical reaction of 23,800 lbs., which we may take equal to 11,900 (putting $\mu = 0{\cdot}5$) leaves a force of 37,600 lbs. actually carried by the hook, corresponding to a stress in the steel of $\frac{37{,}600}{6 \times 3} = 21{,}000$ lbs./in.^{2}.

We do not know whether the hooks would have carried more, but we do at least know that they carried so much.

The constancy of the writer's factors for beams 1 to 8 is specially interesting, as these are cases of superimposed inclined compressions. The author can offer no explanation why beams 13 and 14, which give the lowest factors, should have done so.

Other sets of experimental beams designed and tested by the writer, including a series on plain rectangular beams, and one on T beams, also showed that the writer's formulæ gave more consistent results than either L.C.C. or R.I.B.A., but this paper is already too long to include them.

INDEX

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AND

NOTES

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CONTENTS.

NOTICE.

The War, responsible for so much, is responsible also for the delay in issuing the Transactions. The Council has, however, now decided to furnish Members without further delay with the Papers read during the Session 1916-17. At the same time it has been decided to abandon for the time being the printing of the Discussions, which followed upon the reading of the several Papers.

It may be possible, upon the resumption of normal arrangements, to produce these Discussions in an abridged form.

Intermediate volumes (VI. and VII.) will be published as soon as possible.

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

PRESIDENTIAL ADDRESS

By F. E. WENTWORTH-SHEILDS, M.Inst.C.E.

GENTLEMEN,—I have first to thank you for doing me the honour of electing me as your President. I should like to add that I am conscious of taking office at a time when the work of societies like ours is of national and imperial importance. The European War, horrible and devastating as its effects have been, has produced in this and other countries a wave of energy and enthusiasm for the better study of science and the arts and their application to industry. And this is not to be wondered at. There may be some who say: "Let us concentrate all our thoughts and energies on winning the war, and on nothing else. Science and art are of little importance at the present time." But this is far from being the case. It is true that our first object must be to win the war, and that nothing must stand in the way of achieving it. But we now realize that this is, to a large extent, an engineer's war, and that science and the arts, industry and invention, play a most conspicuous part in it. In every branch of our Army scientific men have been pressed into the service to help with their knowledge and experience. The same remark applies to the new Government Department which provides the munitions of war, and to the many factories and organizations which are helping it. The chapter of history

describing what the engineer, the man of science, and the captain of industry have done during the past two years will, I hope, some day be written. It will certainly be of extraordinary interest. And may I add that the special branch of work fostered by our own Institute will show some very bright achievements.

But there is a special reason why we must regard the work of societies like our own as particularly important at this time. When the war is over, our next object will be to worthily maintain all that we have fought for and to make impossible a repetition of such a devastating struggle. This is a matter which will call for greater and wiser efforts, individual and national, than even the winning of the war will have done. It must not be supposed that all we have to do is to "crush the Germans" and that then our troubles will be over. It is doubtful, to say the least of it, whether any nation can be crushed, and this is certainly not the object we have in view. We are giving our energies, our treasure, and our lives to break the military tyranny of the Central Powers, and to establish the freedom of nations. And when peace is declared we must hold on to the strong position which every day we are gaining. To achieve this we must be ready to solve with energy and wisdom the many and serious problems which peace will undoubtedly bring, and in order to be ready we must prepare for them now. These problems will call for all the best brains that our country can give, and societies like ours will need to put forth all their strength. To-night I want to consider with you one or two of these problems, and how we, as members of the Concrete Institute, can play our part in their solution.

After-war Problems.

When the war is ended we shall find ourselves faced with an enormous financial debt. Even by the end of next April it is estimated that our war loans will have amounted to £3,400,000,000. Consequently, taxation for many years will be extraordinarily heavy. We shall, of course, meet this taxation out of the profits made in industry and trade; but here we shall work

under difficulties, because other countries with whom we have traded in the past will be similarly burdened and will be by no means good customers. Then, again, the delicate machinery of international credit, which was so rudely shattered when war broke out, will take time to repair. Shipping will probably be scarce, and consequently both imports and exports will be delayed. All these and other causes will tend to produce unemployment, with its attendant difficulties; and there is, of course, the danger that these difficulties may be further complicated by the anxiety of the workmen to place all kinds of restrictions on output.

The immensity of the problem involved in resuming, after the war, our usual commercial life will be realized when we consider that Great Britain alone has been diverting the activities of over 10,000,000 men to unproductive and, worse still, to destructive purposes. To restore these activities to normal paths in face of these difficulties will be a task requiring energy, foresight, and wisdom on the part of every member of the community.

And how, then, are these difficulties to be met? In a word, they must be met by economy—economy, not only in the restricted sense of curtailing expenditure and denying ourselves the comforts and luxuries of life, but also in the broader sense of ordering each man's work in the way that Nature's laws direct, so that there shall be the least possible waste and the greatest possible output of things that are really valuable to the nation, to its character, its health, and its happiness.

Economy.

Now, one of the most obvious dictates of economy is that we all should curtail our luxuries and reduce waste of our necessaries. A good many of our luxuries have already gone, and it will be agreed that we are little the worse for their loss. This is especially true of the luxury of idleness. It is good to see that many who before the war enjoyed unearned incomes, and did nothing in return, have developed a sense of citizenship, and would now feel ashamed if he or she were not doing their utmost.

We hope and believe that this spirit of citizenship will live and grow.

Again, Mr. McKenna has pointed out that, comparing to-day with the period 1872-6, when wages were abnormally high, we are now consuming per head per annum, 8 per cent. more wheat, 11 per cent. more meat, double the quantity of tea, nearly double the quantity of sugar, five times the amount of cocoa, and 50 per cent. more tobacco. We may perhaps regard these things—even the tobacco—as necessaries to a healthy life, but are we sure that there is no waste in their distribution and consumption?

Then, again, most of these things are imported. Are we sure that there is no waste in the production and distribution of commodities which we provide in order to pay for them? This is perhaps the most important matter of all. There is no doubt that the nation which will recover most rapidly from the ruinous wastage of war, will be the one which can produce the greatest output of marketable goods at the lowest cost. And how will Great Britain stand in this respect?

I do not want to be numbered among the croakers who decry everything that is British and who say that our education, our science, and our organization are neglected, and that we are a decadent nation. The war itself has given the lie to such a statement. But the war has also taught us that the courageous individualism, and the keen competitive spirit, which has done much to develop our national strength, has its limitations, and that in future it will be necessary to organize our manufactures and trades on more scientific and co-operative lines than ever we have done in the past.

Scientific Research.

For many years we have been dimly conscious that most of our industries could be more economically worked if we gave more attention to scientific and industrial research; but it seemed to need the European war to make us take vigorous action in the matter. It must be owned that previously our achievements in this direction compared unfavourably with those of some other nations. The National

Physical Laboratory was established in 1900; but with a grant of only £4,000 per annum its usefulness was necessarily limited. It is true that our Government has contributed to the funds of the Engineering Standards Committee and of the Imperial College of Science, both of which have done a certain amount of research work. But when we consider that the American Government has been in the habit of subsidizing their Bureau of Standards to the extent of nearly £100,000 per annum, our own national expenditure on such work seems to have been poor.

But in July 1915 a special committee of the Privy Council was formed with a strong Advisory Council, with Sir William McCormick, LL.D. as Chairman, and empowered to spend £25,000 in 1915-16 on scientific research.

This body has already done good work. It has helped researches which had been already undertaken by various scientific societies and private firms, and has encouraged new ones. A complete list of these is given in the Council's Report for 1915-16. One research of great interest to ourselves is the matter of deterioration of timber, metal, and concrete in sea water, which is being investigated by the Institution of Civil Engineers. The Concrete Institute, too, has not been idle, and with the help and co-operation of the professors of several British Universities and technical colleges it has drawn up a most useful programme of tests to ascertain the best way of using various concrete aggregates obtainable in large quantities in all parts of the country. Briefly, the scheme is as follows: Concretes will be made from a variety of aggregates mixed in the familiar proportions of $1 : 1\frac{1}{2} : 3$ and $1 : 2 : 4$. Others will be made equally rich in cement, but with the proportions of sand to stones varied. These concretes will be tested for strength, permeability, and other properties. In this way it is hoped to discover, not only the relative value of various aggregates but also the most advantageous way of using them. It is evident that such a research will be of untold value to the designer, to the builder, and to the community at large.

Our Council have had several interviews with the Government Research Council on this matter, and are

confident that we shall receive assistance from them in this good work. We shall also ask for assistance from those who are interested in this particular industry, and I am sure we shall not ask in vain.

There are, of course, very many other subjects for research which our Institute can usefully promote as soon as our funds and organization will permit. Indeed, they are too many to mention here. But I will briefly allude to one or two which are of more than ordinary interest. One is a line of investigation which has been taken up by Professor Coker, of University College, London, and described by him early this year before the Royal Institution. He has taken advantage of the discovery of Sir David Brewster a hundred years ago, that transparent materials when stressed become doubly refractive.

Professor Coker has devised a most ingenious optical apparatus by which a beam of light passed through a transparent model and also through an arrangement of Nicol's prisms becomes coloured, the colours varying with the intensity of stress applied to the model. In this way the distribution of stress on any section of the model can be accurately determined, and from the information thus obtained the model itself can be improved, so as to make the distribution of stress more even, and waste of material can be thus avoided. At the Royal Institution the lecturer applied his principle to transparent models of such things as an eye bar, a thick cylinder, a cog wheel in gear, and even a roof truss with riveted joints. It is evident that such an aid to the difficult problem of determining the exact state of stress in any part of a structure will be of untold value to the designer.

Again, one of the most important recent investigations on Portland cement has been made by Mr. Nathan C. Johnson, of New York, and published in the *Engineering Record* and in *Concrete*. By means of the microscopic examination of various concretes he has ascertained that, generally speaking, the chemical combination between water and cement in mortar or concrete is exceedingly imperfect, and that only about 25 per cent. of the clinker is properly hydrated. He suggests that an enormous economy could be effected

if this could be improved, as probably the 75 per cent. of uncombined cement is useless except as a pore filler, which for this purpose might with advantage be replaced by a stronger and cheaper material. He further shows that, although this want of proper hydration is probably aggravated by insufficiency of water and of mixing, it is difficult in our present state of knowledge to avoid it. He throws out, however, some suggestions for possible improvements in this respect, and gives results of his own investigations. He points out that one impediment to proper hydration, as revealed by the microscope, is the tendency of the cement particles when mixed with water to cohere into groups, and thus avoid proper contact with the water, owing to the surface tension of the liquid itself. By the addition of certain other liquids, such as alcohol or ether, to the water, the surface tension is reduced, and the cement particles are better dispersed in consequence. The practical advantage of such added liquids is shown in testing the crushing strength of concretes made with and without the addition of such liquids. For instance, with 5 per cent. amyl alcohol added to the mixing water the strength of the concrete was increased about 70 per cent. at two months. It is not yet possible to point to any such substance which can be added to concrete economically, and without fear of detrimental effect, but it is clear that the subject is of the first importance, and calls for further research.

This is the more evident when we consider that the uncombined cement in a concrete is not only wasteful, but may even, under certain circumstances, be dangerous. This is clearly shown in a paper read by Mr. A. H. White before the American Society for Testing Materials. He describes investigations and experiments that seem to show that concrete, if alternately wetted and dried, is made to expand and contract, the net result being a gradually increasing expansion, and that if the process is repeated often enough disintegration may result. This disintegration is explained by him as follows. When water enters into the pores of dried concrete, it combines with cement clinker which has as yet been unaffected. The result is that the concrete is expanded and the pores

filled up, whereupon the action ceases until the concrete is dried again. Then the concrete once more becomes porous, and on re-wetting the expanding action is repeated. It is rarely, of course, that deterioration from this cause becomes serious in practice, but instances of such deterioration are not unknown, especially in the case of marine works and of pavements exposed to weather. They can probably be guarded against by the use of some protective coating, but this, again, is a matter which requires investigation.

The researches to which I have here alluded may all be classed as industrial, in that they may be expected to introduce economies in some industry (the building industry in these cases) within a measurable period. It is important that researches which are more purely scientific and less utilitarian should not be neglected. Their educational value is very great, and, moreover, their application to industry is probably only a matter of time and invention. Witness the case I have just cited of Professor Coker's application of the phenomenon of double refraction. Such purely scientific researches cannot, perhaps, be undertaken by societies like our own. But they ought to be carried on in every important laboratory and encouraged by grant of public funds.

On the other hand, industrial researches—those, namely, which may be expected to produce quick and practical results—should be undertaken under the guidance of men who not only have scientific knowledge, but who are also acquainted with the actual working of the industry for whose benefit the research is needed. To carry this out effectively means co-operation between the laboratory and the works, or, say, between the professor and the manufacturer. It means even more than this—namely, co-operation between the various manufacturers who have hitherto been inclined to look upon each other as rivals, and to conceal from each other any knowledge and experience they may have gained. In this connection it is encouraging to read in the Annual Report of the Advisory Council on Research that " some manufacturers, at any rate, are beginning to realize that their most numerous and dangerous enemies are, not their fellow-countrymen but powerful combinations of manu-

facturers in other countries, supported by every device of rate and tariff that their Governments can provide." And again: "In the numerous conferences we have had with representatives of different industries, we have been impressed with the spirit of co-operation which is growing up, and the willingness to accept our suggestions for the initiation of research for the benefit of the trade as a whole."

Education.

Although a great deal has been done in this country to improve the system of primary education for our working classes, it is evident that more remains to be done if we are to continue to hold our proper place among the nations in social and commercial life. The State now undertakes and insists upon the education of children up to the age of fourteen. Unfortunately, a large number then take up blind-alley occupations, which offer them high wages but no training, and leave them two or three years later almost incapable of learning anything which will be of real value to them in after-life. Even when a boy is apprenticed to a trade at sixteen years of age he is not taught as he should be. It is true that in all our large towns evening classes are established at which he can learn many useful things, but most boys are too tired to derive benefit from lectures after having been at work for ten hours or more. It is generally felt that some better system is needed, under which a lad will continue his education, and not interrupt it just at the time he needs it most. The difficulties in introducing such a system are that it will involve increased expenditure to the parents or to the State, or both, and that it will produce a shortage of labour. To get over the latter difficulty, it has been suggested that we should adopt some such scheme as has been successfully carried out at the School of Pennsylvania, U.S.A. Under this arrangement a four years' course of training is given to a youth after he leaves his primary school at the age of fourteen. The first year is spent wholly in the trade or continuation school, and the boy specializes in those subjects which will best fit him to take up his trade in the shop. The next

three years are given partly to school and partly to shop work. During the time that the schools are in session each boy attends every alternate week, the remainder of the weeks being spent in the shop. The special point of the scheme is that the boys are paired in each trade so that while A is in school B, his mate, is in the shop, and vice versa. The scheme is, in fact, a half-time apprenticeship without the difficulties which attend the work of a single boy working half-time, i.e. for so many hours a day or so many days a week. The work of the two paired boys has been proved by experience to be at least equivalent, as regards value of output, to that of the single boy working full time.

We have not yet outlived the day when education was looked upon as a privilege of the upper classes, and when it was considered that national schools would have a demoralizing influence upon the poorer citizens and produce among them discontent, and even inability to perform the humbler kinds of labour. It is scarcely yet recognized that there is no sort of labour which cannot be made more interesting and more productive by education and scientific training, and again, that if our civilization is to be a real national progress, the development of the mental and spiritual faculties of each of us must be an important item in our national programme. When these facts are clearly recognized, the country will no longer grudge the necessary expenditure on training its boys and girls more thoroughly than it does now, and it is certain that such expenditure will be amply repaid.

Passing to the question of secondary education, every member of our Society will be in cordial sympathy with the movement to emphasize the importance of science in Public School and University education, and to abolish the absurd proportion of time given to the study of Latin and Greek. We do not for a moment, of course, wish to do away with the cultivation of humane letters, but that our two senior Universities should practically compel their students to spend an enormous part of their school hours in learning subjects which the majority of them cannot appreciate, and from which eventually they derive little or no benefit, is a national scandal. It is curious to note that at the important Conference which was held on

this subject last May, under the Presidency of Lord Rayleigh, many masters and professors of the older Universities bewailed the neglect of science in their schools. But the influence of the dead hand seems paramount at Oxford and Cambridge, and probably it will be long before their regulations and scholarships will be arranged in such a way as to induce a larger number of students to take up pure and applied science instead of classics. As a nation we are suffering severely from this neglect of science in the past. Some of our ablest men, who ought to be helping the country as engineers and leaders of commerce, are wasting their energies as lawyers and Members of Parliament. Worse than that, even in this capacity they are quite unable to appreciate the dire necessity for the better cultivation of science, because they know nothing of it themselves. The Report of the Research Council points out that one of the great difficulties of carrying out research work is the scarcity of qualified observers. "The annual number of students," it tells us, "graduating with first and second-class honours in science and technology, including mathematics, in the Universities of England and Wales before the war was only about 530, and of these but a small proportion will have received serious training in research." How is it possible that our Government will take active steps to remedy this state of things so long as their Members and employees are recruited from a class which knows little or nothing of science? Fortunately, the newer Universities are more alive to the importance of science and less hampered by the tradition which has made it the Cinderella of school subjects. Even here, however, there is room for further usefulness. If science is to be really and seriously cultivated in this country, it should be taught at our Universities to every member of our leading professions, to our Civil servants, and also to those who are to take a leading part in commerce and industry of all kinds. But the influence of our Universities should not cease at this point; they should be in constant touch with the life and work of the men they have trained. It is perhaps unfortunate that the professors of our British Universities are in many cases schoolmasters and nothing more. In other countries

it is not considered desirable that students should be taught by a man who does nothing but give lectures. The professor is encouraged to keep in touch with the world of commerce in various ways. He is permitted, or in some cases obliged, to act as consultant on practical work. At one University in America, commercial firms are invited to bring special research problems to be investigated by the College staff, who are bound to secrecy, and, on the other hand, participate in any success which the investigation may produce. Such intimacy between school and business is sure to be of the greatest benefit to both.

Economic Independence.

The war has strongly impressed another idea upon us, namely, that it is consistent with true economy that we should make use of our own national and imperial resources, rather than be entirely dependent upon other nations for the materials and manufactures which we use day by day. "When war broke out," says the Report of the Research Council, "our supply of dyes and glass was practically cut off. We were dependent upon Germany for magnetos, for countless drugs and pharmaceutical preparations, even for the tungsten used by our great steel-makers, and for the zinc smelted from the ores which our Empire produced." I am glad to add that vigorous measures were taken, with liberal assistance from the Government, to remedy this state of things. But that such a thing could happen has given food for thought. Our easy-going theory of the past suggested that we should buy in whatever happened to be the cheapest market, and pay for the goods with something that we could produce more cheaply still. The drawbacks of this practice are now fairly obvious, and, moreover, we are beginning to discover that we can produce many things quite as cheaply as other nations, if we will only take the same trouble.

A very striking instance of this fact is constantly being brought to the notice of the members of this Institute.

In 1913 the value of timber and wood goods imported from our Colonies and from foreign countries

(mostly from the latter) was over £37,000,000. A very large proportion of this timber was used in building construction of all kinds, and yet we know that a great deal of it could be replaced by steel and concrete manufactured at home, and that the resulting structures would be cheaper and more permanent. The price of timber, which is now three times as great as it was in 1913, is forcing this subject upon our attention, and it is safe to say that when the war is over and prices are down to more normal figures, concrete, plain and reinforced, will largely displace its foreign rival, and that every architect, engineer, and builder will be expected to be well acquainted with its use.

In this connection it may be noticed that an important item in the cost of reinforced concrete work is the provision of the formwork, which is usually made of timber. Indeed, this type of construction has been adversely criticized on the ground that it is necessary to erect a timber building before the more permanent structure can be built up, and that the timber so used is of little or no value afterwards, and is practically wasted. There is no doubt that designers could avoid a great deal of this waste by planning their buildings in such a way that the formwork can be used again and again. We have heard a good deal lately about the standard ship, and it has been suggested that shipbuilding could be enormously cheapened by constructing vessels to one or two standard designs. Why should not we have in the same way a standard factory or warehouse in which the spans and scantlings of the beams should conform to a pattern? Such a design would admit of light steel formwork being economically used, and would mean a saving of labour and material, both to the designer and builder. It only requires closer co-operation between all connected with the industry to bring it about, and I think the Concrete Institute would do a good work if it were to lay down such a standard.

Increase of Output.

One of the great difficulties which we have had to encounter in our endeavour to obtain economical work has been the restriction placed by the workmen

themselves through their trade unions on the amount of their output. To Mr. Lloyd George belongs the credit of having induced them to some extent to forgo these restrictions, although they are by no means abolished. Unfortunately, the Government has promised that after the war is over all such restrictions shall be re-established. In the name of common sense let us hope they will not. The attitude of working men in this matter has been foolish and suicidal in the extreme. The result of the policy is clearly seen by comparing our census of production of 1907 with the American census of 1909. In almost every trade the American workman produces two to three times as much as the Englishman. No one would suggest that the American is cleverer or stronger than our own artisan. It is simply that the point of view of the American workman is, or at all events has been, entirely different to that of the British. Our men seem to labour under the mistaken notion that in every workshop or group of shops there is only a limited amount to do, and that if any one man voluntarily increases his day's work, he is depriving some other man of employment. This, of course, is an entire fallacy. Indeed, it may be said that the opposite is the case, and that the industry which can produce the largest output at a given expenditure will be the one which will be able to employ the greatest number of men. On the other hand, many intelligent working men will reply that they are quite aware of this, but that if they use their best efforts to increase production, the employers, by refusing to raise wages and by cutting down piece-work prices, prevent the workman from getting his fair share of the added wealth. Unfortunately, there is some truth in this criticism. The question was dealt with by Mr. Gerald Stoney, F.R.S., in his address as President of the Engineering Section of the British Association at its meeting this summer. He pointed out that at present the most highly skilled men are paid by time, and thus earn considerably less than others of more mediocre attainments who earn by the piece. This is an arrangement which seems to be neither fair to the men themselves nor advantageous to the works as a whole. Again, coming to the question of piece-work prices, Mr.

Stoney points out that it has been customary to cut down the price when a workman's earnings prove to be more than 50 per cent. in excess of what he would make if paid by time. This, too, is unfair and unwise, and the practice is to some extent responsible for the wilful limitation of output. He suggests that piece-work prices should be fixed by a special department, and that they should not be cut down except in extraordinary circumstances, and certainly not because the man happens to be earning good money. There is no doubt that too much stress has been laid in the past by employers on the desirability of keeping wages low, and we are beginning to realize that reasonably high wages are a positive advantage to a manufacturing concern, provided that they are accompanied by a large output.

Better Use of Machinery.

Another economy which needs to be developed in this country is the more extended use of machinery and of better machinery. In many of our workshops the machines are old-fashioned and insufficient, but their owners have hesitated to abolish them for various reasons. Conservatism no doubt has been partly responsible for this lack of enterprise, and perhaps a more important cause has been the difficulty of obtaining a proper output of machinery owing to the lamentable restrictions imposed by the various trade unions, who seem to be possessed with the groundless fear that the use of machinery will lead to unemployment. Be this as it may, our census of production of 1907 compared with that of America in 1909 shows how much more machinery is used in the newer country, their horse-power per man employed being in many cases double our own. It is notorious, too, that the Americans are more ready to replace old machinery by new than we are. The war has done much to change this, and in many workshops new plant has been laid down in order to meet the imperative demand for increased output. The resulting economy in production has been brought home to manufacturers, and it is not likely that the lesson will be forgotten. Even in the building industries, in

which we are more directly interested, the introduction of new machines has been very noticeable. On many works such appliances as concrete-mixers, block-makers, woodworking machinery, and all kinds of elaborate cranes and conveyers are used, where formerly the work would have been done by hand. The advantages both in increased output and in the quality of the work done have been very great, and further invention will undoubtedly extend these improvements.

Scientific Management.

Among the many ways in which science can help industry is the methodical arrangement and teaching of hand labour. For generations the apprentice has been taught his trade by being put to work along with the older hands, so that he may learn by observation, with perhaps some imperfect explanations. The limitations of this system have been shown in a most interesting way by Mr. Winslow Taylor in his book on "Scientific Management." He shows that very careful measurements and records are needed to find out how a man can do any piece of handwork to the best advantage—that is to say, in the shortest time and with the least fatigue. For instance, he describes how by scientific study of the movements involved he was able to train a man to load up pig-iron into railway trucks in such a way that he could easily increase his output nearly fourfold. He describes how the Bethlehem Steel Company used to employ a gang of men, under an excellent foreman, who loaded up 12½ tons per man per day. The movements of the men were studied, and one by one they were taught how to do it more advantageously. The pupil was made to rest at stated intervals, so much so, that less than half of the ten-hour day was occupied in actually carrying his load. The result was that each man was taught to load up 47 tons per day instead of the 12½ tons, and this without any sign of fatigue. The men's wages were raised 60 per cent., notwithstanding which the unit cost of the operation was very considerably reduced.

Mr. Taylor also quotes the investigations of Mr.

Frank B. Gilbreth, of the American Society of Mechanical Engineers, who studied and improved the art of bricklaying. He describes how he arranged the mortar-box and the pile of bricks in such a way that the bricklayer no longer had to step backwards and forwards or to stoop each time a brick was laid. The bricks were sorted and placed with their best edge up by a labourer. The mortar was tempered in such a way that the brick had no longer to be tapped, but could be pressed down to its proper position. Thus the movements made by the bricklayer were reduced from 18 to 5 per brick. With these improvements a well-trained workman could lay 350 bricks per hour, whereas the average speed under the old methods was 120 bricks per hour.

Relations between Employer and Workman.

But even if the workman is scientifically taught his trade, in order that he shall increase his efficiency to the utmost, it is necessary to gain his goodwill, and this perhaps is the most difficult problem of all. A paper full of interest on this subject was read by Sir William Lever at Manchester a month ago. Its author is well known for the large amount of time and thought and money he has spent on the welfare of his workmen, and his remarks are therefore worthy of special attention. He points out that the old idea that labour is merely the paid tool of capital has got to go, and that the attitude of obstinacy and distrust shown by working men towards their employers will not be changed merely by giving them higher wages and better conditions of life. They have shown that under certain circumstances they are capable of high ideals and of great self-sacrifice. This has been remarkably evident in the way they have fought for their country, although many have not as yet applied this spirit of self-sacrifice to the ordinary work of commercial production. This he thinks is largely due to the fact that they have been looked upon as machines rather than as human beings. The arrangement under which all profits or losses goes to capital is not a healthy one, and ignores the psychology of the workman. He suggests that in

every industrial concern there ought to be a ladder from the humblest position to a seat on the board of management, and that both profits and losses should be fairly divided. On both sides there must be give and take. Capital must not expect labour to give up trade unions and the privilege of grumbling for better conditions, and labour must not expect capital to give up control and discipline. In a word, he advocates co-partnership, and deplores the fact that trade unions have opposed it. He regards it, not merely as a system for producing high wages and large profits, but as a spirit which will humanize the relations between man and man, and which by combining the democratic with the individualistic attributes of human nature will result, not only in higher total earnings but in greater efficiency, happier life, and improved mental conditions.

Banking Facilities.

The provision of capital for new undertakings is a matter in which, perhaps, a few of our members can assist, but, none the less, it is of very great importance to us, since the engineer and architect, generally speaking, control its expenditure, and I need make no apology for alluding to it here. It is far from satisfactory to learn that out of £250,000,000 subscribed in this country to capital issues in 1913, less than £50,000,000 was invested at home. During the war, however, an extraordinary number of new factories have been started, and after it is over we shall probably find that the relative importance of our various industries will have altered somewhat, and we shall be called upon to produce many articles for which we have hitherto been dependent upon foreign countries, and also to sell to new customers. Whether as a nation we shall be able to meet this demand will depend to a large extent upon what facilities can be given for the finding of capital for new ventures. Such capital will be required, not only for starting new factories and business concerns, but also for financing sales to possible customers like the Russians and the Italians, who are accustomed to ask for long periods of credit. The facilities offered by our British banks in the way

of providing money for such undertakings have been severely criticized by many authorities, and notably by Messrs. Farrow and Crotch in a little book which they have recently published, called "The Coming Trade War." They compare our modern banks very unfavourably with the so-called German system, and again with the Scotch banks of the eighteenth century, which, they say, did so much to assist the extraordinary commercial and industrial development of that country two hundred years ago. The authors point out that we need a banking system which shall favour, not the richest consumers but the largest employers of labour, which, by giving cash discounts under suitable safeguards, shall enable our merchants and manufacturers to sell in countries where long credit is required, and which shall further assist them with all kinds of information.

In response to such criticism, the Board of Trade has appointed a strong committee, with Lord Farringdon as chairman, to report on the matter, and they have recommended that a "British Trade Bank" be established under Royal Charter, with a capital of £10,000,000. This bank is not to rival, but to supplement, the work of our existing banks, and is therefore not to accept deposits at call or short notice. On the other hand, it is to issue credits to British merchants and manufacturers at home and abroad, even accepting risks, when satisfied that the expenditure is likely to be productive. It is to provide capital for extension of plant, or amalgamation of works, or carrying out contracts. It will provide cash in cases where sales are made to foreign buyers who want long credit. Best of all, it is to organize an information bureau, which will collect facts about the status of foreign firms, new business openings, etc., and which will examine industrial projects. This information will be at the disposal, not only of its customers, but also of Chambers of Commerce and other banks. While independent of Government control, it is to receive recognition and assistance from all British Embassies and Legations, and to act as agent to the Government in cases where they decide to assist certain "key" industries, which must have public funds to bring them into existence.

Even those who are not financial experts can see a good many difficulties in successfully organizing such an institution, but it seems to be agreed that a bank to which the assistance of trade will be a primary object, and the making of safe dividends secondary, is a national need and will make for national economy.

INDUSTRIAL CONSULS.

It will be noticed that the report of Lord Farringdon's Committee, which has just been quoted, develops a suggestion, which has been made elsewhere, that instead of depending so largely, as we do at present, on our Consular service for information about foreign markets, we should establish a similar service organized by and for British men of business. It is certainly notorious that our Consular staff in the past has consisted largely of men who, whatever their other qualifications may be, know little or nothing of commerce. It is also notorious that many of them were unsalaried foreigners, and therefore men without any inducement to encourage British trade. It has been stated that before the war, out of 653 unsalaried consular representatives 268, or 45 per cent., were foreigners, and that these included 44 Germans and Austrians. It would certainly seem that, whether employed by the Government or by such an institution as the British Trade Bank, such representatives should be British-born and men with good business training. In this connection it is interesting to note what has been done by the American Bureau of Foreign and Domestic Commerce, a description of which was recently given in an American magazine by Mr. W. C. Redfield, the Secretary of Commerce to the United States. He tells how the Bureau recently secured to a big smelting and refining company the business of smelting Bolivian tin ores which hitherto had all been treated in Europe. It also succeeded in getting the Spanish tax on American coal withdrawn, with the result that exports to Spain were largely increased. Further, it secured contracts for American firms in Norway, Roumania, and Africa. It has ten commercial attachés in various capitals, all of whom are trained business men, and

who speak the language of the country in which they live, and who spend their whole time in building up American trade. It also engages a large staff of travellers, who specialize in certain industries. Before setting out on their journeys, these travellers consult the principals of the industry they are to represent as to what they can supply, and while on their journey they communicate constantly with them through the Bureau, obtain samples of what is needed abroad, learn the special difficulties that have to be encountered, find out the methods of their competitors, and on their return see the manufacturers at home and give to them all the information they have acquired. It is interesting to note, too, that this Bureau of Commerce not only publishes frequent up-to-date reports, but also owns and manages the American Bureau of Standards, whose magnificent work in industrial research is more or less known to us.

I do not wish to suggest that British manufacturers have been wholly idle in this matter. For instance, the Associated Portland Cement Manufacturers and the British Portland Cement Manufacturers, which together provide about 95 per cent. of the total British export of cement (which in 1913 amounted to 750,000 tons), maintain permanently a representative in South America, where before the war a large market was keenly competed for with Germany. The representative in question is a trained chemist and works manager and good linguist. The same firms also send representatives to Australasia and India, and have organized works in British Columbia, Mexico, and South Africa. Such enterprise is highly commendable, but it is evident that only very large and wealthy firms can manage to keep such an efficient travelling staff, and that if British industries generally are to be enabled to sell their goods steadily new markets must be found for them by some organization encouraged and assisted by the Government. It must be remembered that the Government is to all intents and purposes a third partner in every business concern. Since the war excess profits have been taxed to the extent of 60 per cent., and even before the war incomes which were mostly derived from commercial profits were heavily taxed. Probably such taxation is just, but in return

the Government should see to it that no essential industry lacks capital or a market for its productions.

We have now considered very briefly some of the problems which we shall have to face when war is over, and how they can be solved by the exercise of national economy. We have used the word "economy," not only in the restricted sense of avoiding expenditure, but in its truer and larger sense of establishing the "law of the house," the law under which each man works, not for himself alone, but for the community of which he is a member. We have seen that by using our powers in co-operation, as well as individually, we can do much more than we have in the past to produce those things which are really essential for the health and happiness of the nation and the Empire. We have dwelt on certain lines along which such co-operative efforts may be expected to produce great and beneficent results. We have considered the encouragement of scientific and industrial research, the lessening of our economic dependence on foreign nations, the more liberal use of machinery, the introduction of greater facilities for advancing money, the collection of information about new markets, the closer co-operation between employers and workmen, and, perhaps most important of all, the improvement of our methods of education. All these are matters of vital importance to the nation at this time. Moreover, they are matters in which the Concrete Institute is profoundly interested. In fact, we may say that it is with the concerns of national economy that we have been occupied during the few years of our existence. Economy is a matter that essentially appeals to us. The Charter of the Institution of Civil Engineers describes its members as those who direct the great sources of power in Nature for the use and convenience of man. The racy American says that the engineer is a man who can do for one dollar what any fool can do for two. Both definitions contain a truth and an ideal which appeals to us all.

A glance at our past year's work will illustrate our interest in economy in the best sense. We have had six

General Meetings, at which stimulating and thoughtful papers have been read and discussed. Our Standing and Joint Committees have met regularly to solve such problems as the promotion of research work, the establishment of right relations between architect and engineer, the standardization of measurements, advice about the methods of executing concrete work, and the correct interpretation of the laws relating to steel-framed buildings. All these matters represent an endeavour to introduce economy in building, and when it is remembered that building is one of our largest industries, and that one-tenth part of the nation's industrial output consists of buildings and engineering structures, it will be realized how important is the work which lies before a Society like ours.

Again, we have seen that the Great War has taught us that to be fruitful our economy must include and be inspired by the spirit of co-operation. Our individualism has been a source of immense national strength to us, and I hope we shall never lose it. But we must also cultivate the spirit of mutual service between the State and the civilian, between employer and workman, between one profession and another. Among the members of professions we must include the manufacturers and traders, the captains of industry, from whom we expect great things at this time. The learned professions have been inclined in the past to look down on such men because, they said, their first object was to make profits. But why should it be their first object to make profits? Ruskin long ago pointed out that the merchant's first object should be to provide for his country, and that he should be ready, like a soldier, to die in the attempt to carry out this object if needs be. This may sound too good to be true, but I venture to say that it is right, and that we must expect all our merchants and manufacturers to take this view. The best of them have always done so. Our country will not keep the high place she has regained among the nations unless they do.

The war has had cruel and terrible results. There is not one of us who has not felt the sting of its destructive power. But strangely enough this fearful thing has opened our eyes, and has made us see many truths which had become dim. It has shown us a

vision of what England and the British Empire can and must attain, and every man and every society of men must bear its part in realizing the new ideal. Among those who will help to realize it will, I doubt not, be numbered the Concrete Institute.

THE CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, ETC.

Date of Reading: December 21, 1916.

PILE-DRIVING AND THE SUPPORTING POWER OF PILES

By HENRY ADAMS, M.Inst.C.E., Etc. (Past President).

INTRODUCTORY.

Twenty-five years ago the author read a paper before the Society of Architects upon "Timber Piling in Foundations and other Works." The notes then collected and since added to relate chiefly to timber piles, with which the author has had considerable experience, and upon which much scattered information has already been published; but all driven piles, whatever the material, must behave more or less according to the same laws, and the knowledge gained concerning ordinary timber piles will be likely to enable us to discover the laws and regulate the practice of driving and using piles of other material, including reinforced concrete.

The author may say at once that the repeated inquiries, addressed by the Concrete Institute to those engineers and others who were known to have used reinforced concrete piles, resulted in an extremely small amount of information being furnished of which any use whatever could be made. Such as was given was formulated in a Short Report by the author to the Science Committee of the Institute on November 30, 1911.

Different writers give different formulæ, using the same letters for different purposes, and sometimes omitting to state either what the letters stand for or what units are to be adopted.

In order that the formulæ here given may be easily compared, advantage has been taken of the Standard Mnemonic Notation issued by the Concrete Institute. This notation can be used with any system of units.

By converting the different formulæ into Standard Notation it is made abundantly clear that many formulæ which were formerly considered as divergent are really identical in every particular except notation.

The chaos which has crept into notation is very well exemplified by the curious association of symbols in the Dutch and French formula as quoted by Molesworth.

For example, Molesworth states—

$$L = \frac{W^2 h}{k e (W + M)}.$$

In this equation—

L is the curtailment of the English term "*Load.*"

W is the curtailment of the English term "*Weight* of the ram."

h is the curtailment of the English term "*height*," also the French "*hauteur.*"

k is the English equivalent of the Continental and Greek "kappa"—an international symbol for a *constant.*

e is the curtailment of the French phrase "*enfoncement final du pieu*," or final set of the pile.

M appears to be arbitrary.

Sometimes M is used for the term "*poids du mouton*," or weight of the ram; but in this case M represents "*poids du pieu*," or weight of the pile.

Thus in one simple formula the symbols L and W are translated into English from the French,

h is the same in English and French,

e is left in the original French,

k is international and common to the chief European languages,

M is arbitrary and does *not* mean "*poids du mouton.*"

In English abbreviations this formula would be—

$$R = \frac{W^2}{C (W + W_p)} \cdot \frac{h}{s},$$

where R = Safe *Resistance* of the ground to further penetration by the pile in tons,
C = a *Constant* = 10 for rams lifted by hand, steam or other power winches,
W = *Weight* of the ram in tons,
W_p = Weight of the pile in tons,
h = height of fall in inches,
s = set of pile under the last blow of the ram in inches.

It will be seen that quantities of a like nature such as R, W, and W_p, and also h and s, have been grouped together. Since C represents a pure number, it is clear that R will be in the same units as W.

In French abbreviations the formula would be—

$$R = \frac{P^2}{C\,(P + P_p)} \cdot \frac{h}{e},$$

where R = *Resistance* du pieu en kilogrammes,
P = *Poids* du mouton en kilogrammes,
P_p = *Poids* du *pieu* en kilogrammes,
C = Un *Constant*,
h = *hauteur* de chute du mouton en centimetres,
e = *enfoncement* final du pieu sous un coup de mouton en centimetres.

The Standard Mnemonic Notation issued by the Concrete Institute, so far as relates to the present subject, is as follows :—

Abbreviations.

A = cross-sectional *Area* of the pile.
C = a *Constant*.
d = *diameter* of the pile.
E = *Elastic* modulus of the pile.
F = total *Friction* between the pile and the earth.
f = intensity of *frictional* stress = F/S.
Φ = greater *phi* = *friction* modulus.
θ = *theta* = angle of repose of earth.
g = *gravitational* acceleration = 32·2 ft./sec.².
h = *height* of fall of ram (h is the general symbol for height whatever the unit).
H = *Height* of fall in feet (this symbol is only used for equations employing mixed units of feet and inches), $H = H'$ ft.

$H_{,}$ = the numeric of H = the number of feet in the height of fall.

$$H_{,} = \frac{H}{1 \text{ ft.}}.$$

l = *length* of pile below the ground.
l_a = *length* of pile *above* the ground.
$L = l + l_a$.

$$M = \textit{Momentum} = \frac{W}{g} v = m v.$$

$$m = \textit{mass} = \text{inertia} = \frac{W}{g}.$$

P = total estimated *Pressure* on the pile.

(general) p = intensity of *pressure*.

(special) p = intensity of bearing *pressure* when so specified in the context.

Q = a *Qualifier* (i.e. a coefficient).

q = a *qualifier* (i.e. a coefficient).

R = working statical *Resistance* of the earth to further penetration of the pile.

r = a *ratio* as defined by the context.

S = *Superfices* of a *pile* = the *surface* in frictional contact with the earth (i.e. in the case of a square pile $S = 4\,d\,l$).

s = average *set* of the pile under the last few blows = $s_{,}$ in.

$s_{,}$ = the numeric of s = the number of inches in the *set* of the pile $\left(s_{,} = \frac{s}{1 \text{ in.}}\right)$.

U = *Ultimate* statical resistance of the earth to further penetration of the pile.

W = *Weight* of the ram.

W_p = *Weight* of the *pile*.

Note as to Units.

Unless otherwise specified—

(1) d, h, l, l_a, and s are in like units of length.

(2) A and S are in like units of area—(units of length)^2.

(3) F, P, R, U, W, and W_p are in like units of force.

(4) E, f, p, F/S, and R/A are in like units of intensity of force = units of force divided by units of area.

(5) C, Φ, Q, q, and r are pure numbers or pure ratios between like magnitudes.

$H_{,}$ is a *number* of feet in the height of the fall of the ram.
$s_{,}$ is a number of inches in the *set* of the pile.

The last two symbols only occur in formulæ involving the use of mixed units of feet and inches.

As the mnemonic abbreviations represent real magnitudes irrespective of the unit of measurement, the sizes of the units selected in each case have been placed against the particular formula and not against the general list of abbreviations, which are applicable to any or all units, whether Metric or British, and whether tons or pounds, feet or inches.

Driving Piles.

The smallest piles are driven by a "beetle," which is a heavy long-handled wooden mallet weighing about half a hundredweight. For driving larger piles up to 10 in. square an apparatus called a ringing engine is used. It consists of a light framework with a pulley at the top and a rope over it, connected at one end to a ram weighing from 3 to 5 cwt., and at the other end attached to four or more short ropes or tails for pulling by hand power directly applied. One man is allowed to each 40 lb. weight of ram, and two can hold on to one tail. By this means a rapid succession of blows is given with a light fall. The ram is in some districts called a tup, or trip, and in London is very often called a monkey, but the monkey is properly only the slip-hook that runs up and down to lift and drop the ram in the larger apparatus.

Ordinary piles are driven by what is called a "pile engine." It is virtually a large hammer, the weight being arranged to fall freely and strike a succession of blows upon the head of the pile. It consists of a tall framework, with vertical guides on the face to keep the hammer or ram in a direct line with the head of the pile. The base of the pile engine is placed just above the finished level of the pile head, so that in driving long piles a high framework is required. It is, however, sometimes impossible to get the framework high enough for this, and it is then set 6 or 8 ft. above the finished level, and a punch, dolly, or follower, of hard wood, hooped at both ends, is used on the head of the pile when it gets as low as the base of the frame, but the blow is not so effective and the method should be avoided when possible; it is said to reduce the effect of the blow one-third, more or less, according to the rigidity of the material. Becker states that the most convenient weight in pounds for a dolly is—

$$\sqrt{(W + W_p)}, \text{ where } W \text{ and } W_p \text{ are in pounds} \quad . \quad (1)$$

but there does not seem any advantage in having a rule of this kind, even if suitable, which this is not, as the size will generally depend upon circumstances other than weight of ram and pile. Nothing could be more simple or suitable than to use a piece of the pile timber, cut to the length required and hooped both ends, but an ideal dolly would be of oak 9 in. square and hooped square at both ends. . . . (2)
The length of a pile is generally determined by the local conditions of site and soil, the sectional area chiefly upon the load it has to sustain, usually the ratio

$$\frac{L}{A} = \frac{1}{4} \text{ to } \frac{1}{8} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

but no general rule can be laid down, as it depends to some extent upon the unsupported length above-ground.

In ordinary cases the ram is raised by a crab-winch, worked by three or four men, and is connected to the rope or chain for lifting by an intervening slip-hook or "monkey," so that it can be suddenly disconnected and allowed to fall freely when it has reached the required height. In the usual way the ram can only have a small fall at the commencement of driving because of the height of the pile, but the resistance is also small at starting, and as the pile goes down the fall can be increased. If it is then desired to retain a uniform fall, the rope which releases the trigger of the monkey is made fast to the head of the pile, and is by this means pulled automatically at a constant height above the pile whatever its position may be. Where there is much piling to be done, the hoisting of the ram is usually effected by a small steam-engine attached to a vertical boiler and placed close alongside. By throwing a clutch out of gear the chain is rapidly lowered to enable the ram to be again connected, and much time is saved. Piles penetrate quicker and more easily when driven with a minimum interval between each blow, as the soil has then less time to settle and adhere to the pile.

An interesting account is given in the *Journal of the American Society of Engineers* (1892) of how, when a certain swamp had to be crossed by a railway, the attempt to find a sound bottom by the longest piles resulted in failure. The piles could easily be driven out of sight by the ordinary method. It was accidentally discovered, however, that if every pile was merely sunk into its place by the added weight of a heavy man and left there for a while, it would answer every

purpose, and, as a matter of fact, trains are said to be actually running over viaducts that were piled in this peculiar manner.

In a paper read before the American Society of Civil Engineers by Prof. W. H. Burr we have an illustration of pile-driving of a diametrically opposite character, viz. driving through rubble stone. Nineteen experimental piles were driven through stone-filled crib work. The crib was 35 ft. deep, and the piles, from 52 ft. to 60 ft. long, were shod with chilled cast-iron points, held by straps spiked to the pile. The ram used in the driving weighed 3,500 lb., and the greatest number of blows per pile was 350. There was no difficulty whatever in driving the piles, and there were no failures. In the discussion following the reading of the paper several other particulars of pile-driving through loose rocks were given. The shoe used in the work is conical, the angle of the cone being about 60 degrees, the base of the cone is flat, and it is secured to the pile by a pin cast as part of the shoe and let into the end of the pile. This type of shoe is said to have proved very satisfactory in practice.

Pitching and Driving.

In pitching a pile care must be taken that it is started in the right place, as it cannot be shifted, but if the point is not truly in line with the axis of the pile, or gets pushed to one side by meeting an obstruction before it has entered very far, the lower end of the pile will be drawn over to the side to which the point leans. In spite of the greatest care they will sometimes be found slightly out of position, and they have then to be drawn back into place by chains, twisted like a surgeon's tourniquet, while being bolted to the other timbers. If it be necessary for them to be scarfed, the upper portion can be adjusted by cutting the scarf a little out of line to suit. All piles are not required to be vertical ; in building jetties the outside row of piles is often doubled, the outer pile being a raking one, at 15 to 30 degrees from the vertical, for increasing the stability, acting like a buttress. At the corners of jetties the outer piles are usually raking both ways, say about 15 degrees from the vertical. When necessity arises piles may be drawn from the bed of a tidal river by lashing empty barges to them and letting them lift by the tide. Against a river wall or round the foundations of a bridge piles should be sawn off by a diver as low as he can get at them in preference to drawing them, to avoid any risk of scour and undermining of the foundations taking place. On land a pile

may be drawn by lashing a short piece to the top and then prizing it up by another baulk used as a lever, or by a pair of powerful jacks.

WEIGHT OF RAM.

One of the most interesting questions in connection with pile-driving is the proportion between the weight of ram and the fall to produce a given result. The ram usually weighs from 5 to 30 cwt., and is allowed to fall, say, from 6 to 20 ft. Upon a superficial consideration it would seem that a ram of 5 cwt. falling 20 ft. would produce the same result as a ram of 20 cwt. falling 5 ft., as they would both have 5 ft.-tons

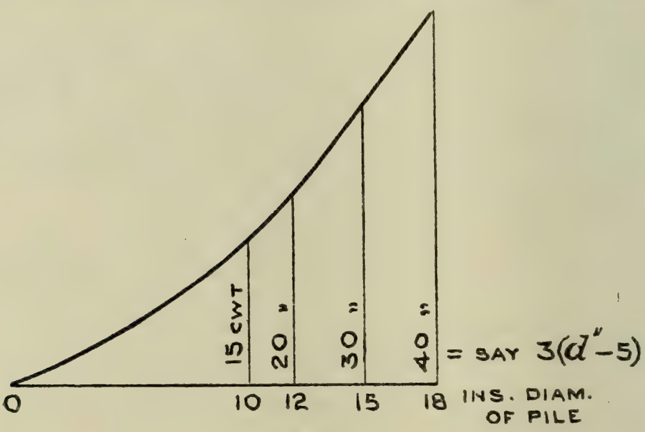

FIG. 1.

energy, but the proportion of the total energy (W h) which is usefully expended in sinking the pile depends *inter alia* upon the ratio of the weight of ram to the weight of the pile. Some of the total energy is always wasted.

A light ram with a long fall will not have the same effect as a heavy ram with a short fall. In practice it is found that with too great a fall the effect of the blow is to bruise and "broom" the head of the pile, or to shiver the timber instead of to force it downwards. A heavy ram, producing the same effect in distance driven as a light one with greater fall, does less injury to the pile. Dobson says, "In working with a fall from 12 to 20 ft. it is common for every tenth pile to be more or less shaken." Of course he meant to say "one pile in ten." It is as if the top of the pile were driven down

while the bottom remained stationary, owing to the inertia of the mass of the pile. He gives a table for weight of ram desirable for various sizes of piles, this is laid down to a curve in Fig. 1, and is given very nearly by the formula—

$$\text{Weight of ram in cwt.} = 3\ (\text{diam. pile inches} - 5) \quad . \quad (4)$$

Hurst's "Tredgold," p. 292, says, "Piles from 10 to 14 in. diameter require to be driven with a ram of from 1,000 to 1,700 lb. weight. Sheeting piles require a ram from 500 to 900 lb. weight." . . . . (5)

Molesworth's rule for weight of ram (p. 117, 21st edition) translated into standard notation is—

$$W = W_p\left(\frac{H \times W_p}{5\,A\,L} - 1\right) \quad . \quad . \quad . \quad . \quad (6)$$

W and W_p being in pounds; but this does not seem a reasonable formula as the weight of ram required would increase with the available fall instead of the reverse, and in many cases it would give a minus value. Inverting the fraction and omitting the -1, the result would be more reasonable.

$$W = W_p\left(\frac{5\,A\,L}{W_p\,H}\right) \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Hurst says ("Pocket Book," 13th edition, p. 150), "A light ram with a high fall is best for clay soils, and a heavy ram with a low fall for sandy soils."

Perhaps a simple and suitable rule would be to make the weight of ram equal to $1\frac{1}{2}$ times the weight of pile. . . . (8)

Goodrich gives weight of ram not less than weight of pile—

$$W \not< W_p \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

but if made only equal it would be rather light.

Hurtzig gives the rule—

$$W_{\text{cwt.}} = \frac{L\,A}{448} \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

The weights of ram most favoured by railroad engineers, representing 37 roads, and by 14 other engineers and

*

contractors, according to the *Engineering Record*, May 1, 1909, are—

$$2{,}500 \text{ and } 3{,}000 \text{ lb.} \quad \ldots \ldots \quad (11)$$

as shown by data in the report of the Committee on Wooden Bridges and Trestles of the American Railway Engineering and Maintenance of Way Association, but this result ignores the size or weight of pile. Out of 57 replies to the question as to type of ram, 43 preferred drop hammers, 9 preferred steam hammers, and the others used both.

For the first few blows the pile goes down a considerable distance, which gradually becomes less at each blow until the resistance is so great that it will not go any farther, or, as it is technically called, "refuses." Were the material of the pile perfectly rigid and inelastic, the impact, however slight, would produce an infinite pressure, but the material is very elastic, and so the fibres of the pile are compressed without the point going any farther, and the amount of this compression and the elasticity are strikingly shown in the rebound of the ram when the pile refuses. Generally the driving is stopped when the "set," or distance driven by the last blow, does not exceed ¼ in., or the sum of the last three blows 1 in., but of course for this to be any measure of the supporting power the weight of the ram and height of fall must be in some recognized proportion to the dimensions and weight of the piles.

Rankine says, "According to some of the best authorities the test of a pile having been sufficiently driven is that it shall not be driven more than ⅕ in. by 30 blows of a ram weighing 800 lb. and falling 5 ft. at each blow," and there is probably no one who would dispute the fact that if it stood this test it had been sufficiently driven, many would consider that it had been over-driven and that valuable energy had been wasted. The tendency of late years has been to increase both the weight of the ram and its fall, especially where heavy duty in supporting power is required.

The Supporting Power of Piles.

The sustaining power of a pile depends chiefly upon three circumstances—

(1) The resistance at the point or shoe to further penetration,
(2) The friction of the earth on the sides of the pile, and
(3) The strength, as a column, of the pile above the ground or above the firm sub-soil.

Although it would seem that these are simple elements, each of which could be fairly estimated, the complexity of the case is shown by the numerous formulæ which give results ranging up to about 450 per cent.

The first two factors are usually combined into one formula founded chiefly upon experiments. The best-known formula is that of Major Sanders, U.S. Eng., quoted by Rankine and Molesworth—

$$R = \frac{W\,h}{8\,s} \quad . \quad . \quad . \quad . \quad . \quad (12)$$

This equation is unitally complete, h and s are in the same units of length, and R will be in the same units as W.

For example, if W is in tons, then R will be in tons.

This is the same as calling the safe load one-eighth of the mean resistance to the energy of the blow, assuming it to be expended entirely in penetration without compressing the pile. Although this formula may be good enough for ordinary cases, it can at best be only approximate, and it will be instructive to compare it with other formulæ which have been given from time to time.

Mansfield Merriman's "American Civil Engineer's Pocket Book" gives the above formula with a constant of 6 instead of 8, making the translation—

$$R = \frac{W\,h}{6\,s} \quad . \quad . \quad . \quad . \quad . \quad (13)$$

In Berg's "Safe Building" the same formula is given with a constant of 10 instead of 8, and a note is added, "Where there is the least doubt about the stability of the pile use $\frac{3}{4}$ of result, and if the piles drive very unevenly use only $\frac{1}{2}$. The $\frac{3}{4}$ result would then make the formula—

$$R = \frac{3}{4} \cdot \frac{W\,h}{10\,s} \quad . \quad . \quad . \quad . \quad . \quad (14)$$

This equation is also unitally complete, h and s are in the same units, and R will be in the same units as W.

A. M. Wellington's formula (*Min. Proc. Inst. C.E.*, cxv. p. 317), known also as the *Engineering News* formula, is—

$$R = \frac{2\,W\,H}{s + 1},$$

this presumes a safety factor of 6. The preferable form will be—

$$R = \frac{W\,h}{6\,(1 \text{ in.} + s)} \quad . \quad . \quad . \quad . \quad (15)$$

This equation is only unitally complete when h and s are in inches, as the constant within the brackets in the denominator is really 1 in. and not a pure number. The expression (1 in. + s) is written instead of (s + 1 in.) so as to conform with the general rule as to algebraic form, by which the constants precede the variables. The *Engineering News* and the *New York Building Code* both give this rule in a form which is unitally defective.

This equation may also be written in the form—

$$R = \frac{2\,W\,H_{\prime}}{1 + s_{\prime}},$$

where $H_{\prime}$ is the *number* of feet in the height of fall and $s_{\prime}$ is the *number* of inches in the set of the pile. R and W will be in the same units.

McAlpine's formula (*Min. Proc. Inst. C.E.*, xxvii. p. 280) is—

$$U = 80\,(W + 0{\cdot}228\sqrt{H_{\prime}} - 1 \quad . \quad . \quad (16$$

In this equation $H_{\prime}$ is the number of feet in the height of fall and U will be in the same units as W.

The safe load is assumed to be one-third of this for a dead load, or one-tenth if subject to vibration.

Trautwine's "Pocket Book" gives the following formula—

$$U = \frac{51{\cdot}5\,W\sqrt[3]{H_{\prime}}}{1 + s_{\prime}} \quad . \quad . \quad . \quad . \quad (17)$$

This equation is only unitally complete when $H_{\prime}$ is the number of feet in the *height* of fall, and $s_{\prime}$ is the number of inches in the *set* of the pile.

Factor of safety according to soil from 2 to 12.

Weisbach's formula (quoted by Haswell, *Min. Proc. Inst. C.E.*, cxv. p. 317) is—

$$U = \frac{W^2}{(W + W_p)} \cdot \frac{h}{s} + W + W_p \quad . \quad . \quad (18)$$

This form is unitally complete. U will be in the same units as W and W_p, and h and s will be in the same units whether feet, inches, or centimetres.

Note.—That when $W = W_p$ Weisbach's equation becomes—

$$U = \frac{W\,h}{2\,s} + 2\,W.$$

Mason's formula (quoted by Haswell) is the same as that of Weisbach with the latter part omitted as being of small value.

Tarn gives Weisbach's formula similarly with a factor of safety of 5.

Ritter's formula is the same form as Weisbach's, viz.—

$$W = \frac{h}{e} \times \frac{Q^2}{Q+q} + Q + q,$$

translated into our notation it is identical with that of Weisbach—

$$U = \frac{W^2}{W + W_p} \cdot \frac{h}{s} + W + W_p \quad . \quad . \quad . \quad (19)$$

He recommends a safety factor of 10.

W. Baker (Chief Engineer L. & N.W. Railway) used the formula—

$$U = \frac{5}{6}\left(\frac{W}{W + W_p}\right) \times \frac{h}{s} \quad . \quad . \quad . \quad . \quad (20)$$

with a factor of safety of 10.

Rankine gives a formula (based upon Airey's formula in Whewell's "Mechanics") which takes into account the resilience of the pile. In our notation it is—

$$U = \sqrt{\left\{\frac{4\,E\,A\,W\,h}{L} + 4\left(\frac{E\,A\,s}{L}\right)^2\right\}} - 2\left(\frac{E\,A\,s}{L}\right) \quad (21)$$

This equation is unitally complete—

U and W are in the same units [tons, for example].
h, L, and s are in the same units [inches, for example].
A is in the same units as l^2 [square inches, for example].

E is in the same units as $\frac{W}{A}$ [tons per square inch, for example].

Factor of safety = from 3 to 10.

Rankine also states that the actual load applied varies from 200 lb. in soft ground to 1,000 lb. on reaching firm ground, per square inch sectional area of pile. . . . . (22)

Rankine's "Useful Rules and Tables" gives the ordinary working loads on the heads of piles in foundations as follows—

On piles driven till they reach firm ground—

$$0{\cdot}45 \text{ ton per sq. in.} \quad \ldots \ldots \quad (23)$$

On piles standing in soft ground by friction—

$$0{\cdot}09 \text{ ton per sq. in.} \quad \ldots \ldots \quad (24)$$

Ordinary values of greatest load which piles will bear without sinking further—

$$0{\cdot}9 \text{ to } 1{\cdot}35 \text{ tons per sq. in.} \quad \ldots \quad (25)$$

Molesworth (p. 116, 21st edition) quotes a Dutch rule for the load a pile will bear—

$$U = \frac{W^2}{(W + W_p)} \cdot \frac{h}{s} \quad \ldots \ldots \quad (26)$$

This rule is unitally complete, and it does not matter whether the units are Dutch or Russian or English, U will be in the same units as W and W_p, and h and s will be in the same units. The Dutch rule when written in our notation is seen to be based on Weisbach's and identical with Mason's.

Factor of safety = 10 for ordinary and 6 for steam pile-drivers. Limit of driving in sand, 15 ft.

E. P. Goodrich, C.E., U.S. Navy, gives the following rule: Weight of ram not less than weight of pile, fall of ram between 15 and 25 ft., ultimate supporting power immediately after driving, with a possible error of 10 per cent.

$$U = \frac{10}{36} \cdot \frac{W\,h}{s} = \frac{W\,h}{3{\cdot}6\,s} \quad \ldots \quad (27)$$

Haswell's formula (*Min. Proc. Inst. C.E.*, vol. cxv. p. 318)—

$$U = 4\,W\sqrt{(2\,g\,H)} = 32\,W\sqrt{H} \quad . \quad (28)$$

This rule appears to be based on the same error as Dobson's, referred to later. The equation does not contain any reference to the kind of ground into which the pile is driven, as no set is taken account of.

Factor of safety = 3 to 6 according to circumstances.

A. C. Hurtzig gives a rational formula, which takes the form of a quadratic. The simplest expression in standard notation is—

$$U = \sqrt{\left\{\frac{125\,A}{L} \cdot W\,H + \left(\frac{125\,A\,s}{24\,L}\right)^2\right\}} - \frac{125\,A\,s}{24\,L} \quad (29)$$

This equation is unitally incomplete.

Nystrom's formula as quoted by Haswell will read—

$$U = \frac{W^3}{(W + W_p)} \cdot \frac{h}{s} \quad . \quad . \quad . \quad . \quad (30)$$

with a factor of safety of 6.

This equation is unitally complete. U will be in the same units as W and W_p, and h and s are in the same units.

Note.—h/s is the ratio between two actual lengths, and this ratio exists independently of any particular set of units.

Note that when $W = W_p$ then Nystrom's equation becomes—

$$U = \frac{W\,h}{4\,s}.$$

A simple static method of estimating the bearing power of piles is given in the new edition of Rivingtons' "Notes on Building Construction," vol. i. p. 23 (and by other authors), where the safe load on pile in pounds is given as—

$$R = A\,p + S\,f. \quad . \quad . \quad . \quad . \quad (31)$$

f = resistance caused by *friction* of earth against sides of pile (varying from 150 to 600 lb. per sq. ft.).

S = area in square feet of the *surface* of the pile in friction with the earth.

p = bearing *pressure* or resistance to settling, determined by the bearing power of the earth (2,000 to 10,000 lb. per sq. ft.).

This is apparently taken from W. M. Patton's "Treatise on Civil Engineering" (Wiley & Sons, 1895, New York), where the bearing power of pile in pounds is given as—

$$\mathrm{A}p+\mathrm{S}f \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (32)$$

p = *pressure* or bearing power of soil in pounds per square foot.
A = sectional *area* of pile in square feet.
f = *frictional* resistance in pounds per square foot between earth and timber.
S = area of *surface* in contact in square feet.

In soft soil...	... ...	$p=0$	$f=150$
In ordinary clay ...	...	$p=3{,}000$	$f=300$
In compact clay ...	...	$p=5{,}000$	$f=300$
In sand and gravel	...	$p=5{,}000$	$f=500$

With reference to Nos. (31) and (32) it will be seen that an independent estimate is required of the nature of the soil in order to fix the constants, but the best test of the soil is the set of the pile with the last four blows, and therefore the formulæ based on the latter are more likely to be reliable.

Howe's formula is somewhat similar to the last two, and may be the original (bringing in Rankine's safe pressure on foundations)—

$$\mathrm{R\ lb.} = \mathrm{A}\ l_a\ \mathrm{W}_s\left(\frac{1+\sin\theta}{1-\sin\theta}\right)^2 + 4\ l_a f\sqrt{\mathrm{A}} \quad . \qquad (33)$$

W = *weight* of soil in pounds cubic feet.
A = square feet sectional area.
f = safe resistance to *friction* lb. per sq. ft., say 150 for mud, 300 ordinary soil, 500 Thames ballast.

In sinking bridge cylinders it is found that the skin friction is usually from $2\frac{1}{2}$ to 6 cwt. per sq. ft.

D. K. Clark (Law's "Civil Engineering," p. 49) says that a pile 12 in. square, 20 ft. in ground, will bear—

9 tons in ooze or muddy sand,
12 tons moderately compact clay,
25 tons hard clay,
80 tons if it reaches hard gravel,

but this assumes the pile to be so supported throughout its length as not to be free to bend. . . . . (34)

In Newman's "Earthwork Slips and Subsidences" the frictional resistance of timber piles is stated to be less through wet soils than dry, in the following proportions—

In sandy gravel 5 to 10 per cent. less.
In sand about 12 per cent. less.
In sandy clay or gravelly clay about 40 per cent. less.

Experiments with cast-iron piles by McAlpine gave about ½ ton per sq. ft. of surface as the supporting power from friction when sunk 20 to 30 ft. in rocky gravel. He considers it would amount to 3 tons per sq. ft. in fine earth, but this seems to be an extravagant assumption. In experiments made previous to the sinking of concrete piles for the works of the Vienna-Danube Sand Dredging Company in 1909 it was found that the frictional resistance was about 14·19 lb. per sq. in. of surface = 2054·36 lb. per sq. ft., or just over 18 cwt.

G. P. Bidder was of opinion that "in clay or wet soils it was not advisable to trust a greater weight than 12 tons upon each pile, but in gravel there was scarcely any limit to their vertical bearing strength." . . . . (35)

French engineers (*vide* Berg's "Safe Building") allow a pile to carry 50,000 lb. provided it does not sink perceptibly under a ram falling 4 ft. and weighing 1,350 lb., or does not sink ½ in. under 30 blows. . . . . (36)

The following rule has been given for steam driving. When a pile is driven a feet vertically into the ground by n blows of a steam hammer fastened to the head of the pile, p being the mean pressure of the steam in pounds per square inch, d the diameter of the piston in inches, l the length of the stroke in feet, W the weight in pounds of the moving part of the hammer, W_r the weight of the pile and the fixed part of the steam hammer attached to it in pounds, the mean resistance of the ground in pounds will be—

$$W_r + W + \frac{nW}{W + W_r}\left(W + \frac{1}{4}d^2 p\right)\frac{l}{a} \quad . \quad (37)$$

For the ultimate resistance, the result of, say, the last ten blows may be taken.

G. F. Stickney, M.Am.Soc.C.E., gives the following instructions where test piles are driven to determine the length required in foundations of structures, the safe load will be determined by the formula (in our notation)—

For drop hammer—

$$R = \frac{2\,W\,H_{\prime}}{1 + s_{\prime}} \text{ [see also (15)]} . \quad (38)$$

For steam hammer—

$$R = \frac{2\,W\,H_{\prime}}{0{\cdot}01 + s_{\prime}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$$

These equations are unitally complete, since R will be in the same units as W, and $H_{\prime}$ and $s_{\prime}$ are pure numbers, although they are derived from mixed units of feet and inches.

Kreuter's formula for test piles is given in the *Min. Proc. Inst. C.E.*, cxxiv. p. 373 as follows—

h_1 = height of fall of ram during first test series of blows.
h_2 = height of fall of ram during second test series of blows.
s_1 = average set due to one blow of first series.
s_2 = average set due to one blow of second series.

$$U = W\frac{(h_1 - h_2)}{s_1 - s_2} \quad . \quad . \quad . \quad . \quad (40)$$

This equation is unitally complete.

U will be in the same units as W.

Also h_1, h_2, s_1, and s_2 are in the same units, whether Metric or British, and whether feet or inches.

The piles to rest three days between each test. Heads to be sawn off square before test. This rule appears to have been given first by G. J. Morrison (*Min. Proc. Inst. C.E.*, xxvii. pp. 313–14).

R. P. Brereton made experiments up to breaking-point on fir piles of various lengths and 12 in. square. Fig. 2 has been constructed from the table in Law's "Civil Engineering." The average would appear to give a result as follows: r = ratio of length to diameter, then—

$$U = (120 - r) \text{ tons} \quad . \quad . \quad . \quad . \quad (41)$$

with safe load of, say, one-fourth when stayed at 10 diameters' interval; but it should be noted that the experiments here quoted appear to have been upon the unsupported length as columns, and not upon the bearing power as piles.

Another formula by Lieut. Sankey for the strength of fir posts is the following—

Cwt. per square inch to crush post $=\left(\dfrac{10{,}000}{500+r^2}\right)$ cwt. . (42)

Factory of safety = 5.

An approximate formula of the author's for maximum safe pressure upon a fir pile as a column—

$$P=\left(\frac{300}{r}\right)\text{ tons} \quad . \quad . \quad . \quad . \quad (43)$$

r being the ratio of unsupported length to diameter.

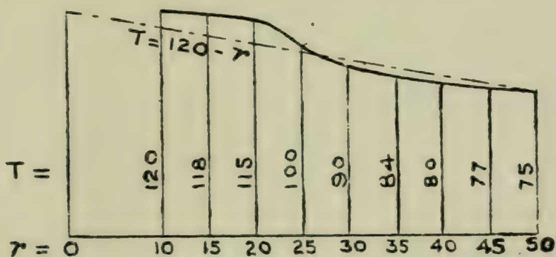

FIG. 2.

From experiments by Kirkaldy upon baulks 13 in. square and 20 ft. long, Riga crushed with 148 tons and Dantzig with 138 tons. . . . . (44)

Rankine gives for short fir posts, i.e. under 20 diameters long, crushing load = 6,000 lb. per sq. in., and factor of safety = 5, therefore safe load = , say, $\frac{1}{2}$ ton per sq. in. . . (45)

A formula founded on Gordon's, and agreeing fairly well with experiments for long fir posts ends flat is—

$$\text{Ultimate crushing load}=\frac{(3{\cdot}2\text{ tons/in.}^2)\,A}{1+0{\cdot}008\,r^2} \quad . \quad (46)$$

By this formula the baulks experimented upon by Kirkaldy would crush with 145 tons.

A common rule for safe dead load on each pile is 5 tons per sq. ft. of cross-section in soft ground, or 1 ton per in. side of square piles in firm ground. . . . . (47)

The New York Building Regulations permit a load of 20 tons per pile, but the size is not specified. . . . . (48)

The author believes they use Wellington's formula (15) for safe load.

Haswell (*Min. Proc. Inst. C.E.*, cxv. p. 318) says, "In deciding upon a factor of safety in a formula for pile-driving, the following elements must be considered: The friction of the machine; the resistance of the atmosphere to the fall of the ram and the cushioning on the head of the pile, however square it may be dressed off; the want of verticality both in the fall of the ram and in the plane of the pile, and the consequent lateral vibration; the inertia; the vibration and condition of the soil. Were all the conditions known definitely and allowed for, a factor of safety of 2 would be ample, but as the formulæ do not take account of all the conditions a larger margin is necessary. In some ascertained supporting powers recorded by Trautwine they were found to be from 2·3 to 3·7 times greater than given by the formulæ.

There seems to be no general rule as to the factor of safety it is desirable to adopt, the practice appears to vary from 2 to 10, the former being suitable for dead loads and the latter for live or vibrating loads. Intermediate factors would be produced with varying proportions between the dead and live loads. Obviously unless the ultimate resistance given by the formula is reliable, the resulting factor is unknown.

In Dobson's "Foundations and Concrete Works" (Weale's Series) we are told that "of the comparative effect of impact and pressure in driving piles we as yet know nothing, and the question is so complicated, from the great number of points that have to be taken into consideration in reducing the results of experiment into a definite form from which some rule for our guidance might be obtained, that we can only lay down in general terms the following empirical rule that in *ordinary cases* if a pile will safely resist an *impact* of a ton it will bear without yielding a *pressure* of $1\frac{1}{2}$ tons," and he gives $W v$ = impact, therefore safe load $= 1·5\ W v$.

Since—

$$v = \sqrt{(2 g h)} = \sqrt{(2 g)} \sqrt{h},$$

we have—

$$\begin{aligned} 1·5\ W v &= 1·5\ W \sqrt{(2 g)} \sqrt{h}, \\ &= 1·5 \sqrt{(2 g)}\ W \sqrt{h}, \\ &= 1·5 \times 8\ W \sqrt{h}, \\ &= 12\ W \sqrt{h}. \end{aligned}$$

We then obtain the equation—

$$R = 12\,W\sqrt{h} \quad . \quad . \quad . \quad . \quad . \quad (49)$$

Dobson is, however, wrong in his theory ; he assumes that the force of a blow is measured simply by the product of the weight into the velocity, and this assumption leads him to conclude that a 1-ton ram with a fall of 16 ft. will have the same effect on the head of a pile as a 2-ton ram falling 4 ft., while the former takes double the expenditure of power to raise it. In other words, he says $f = m\,v$, instead of $f\,t = m\,v$, which is the well-known formula where f = force, t = time, m = mass $= \frac{w}{g}$, v = velocity. That is, a force f, acting for time t, will move a mass m, with a velocity v, but action and re-action are equal in magnitude but opposite in direction, therefore a mass m moving with a velocity v, and expending its energy in time t, will produce a mean pressure

$$f = \frac{m\,v}{t}.$$

Weight = product of mass into force of gravity, or $W = m\,g$, therefore $m = \frac{W}{g}$, or $f = \frac{W\,v}{g\,t}$, instead of $f = W\,v$ as Dobson puts it. The same error is made by Molesworth, p. 117, and a table of results is given assuming that the force of the blow varies as the square root of the fall instead of directly as the fall. The product $m\,v$ gives the momentum in its original sense. It is also known as *quantity of motion*, but it cannot be compared with force or pressure unless time be taken into account.

A worked example will perhaps make this clear—

Let $W = 0{\cdot}25$ tons, $h = 20$ ft., S = set at head of pile $= \frac{3}{4}$ in., then

$$v = \sqrt{(2\,g\,h)} = 35{\cdot}7 \text{ ft. per sec.,}$$

but v varies from 35·7 to 0 while passing through the $\frac{3}{4}$ in., therefore mean $v = \frac{35{\cdot}7}{2} = 17{\cdot}85$ ft. per sec., and $\frac{3}{4}$ in. $= \frac{1}{16}$ of a foot, therefore time occupied in passing through $\frac{3}{4}$ in. $= 1/(17{\cdot}85 \times 16) = 1/285$ of a second. Then by formula—

$$f = W\,v/g\,t = (0{\cdot}25 \times 35{\cdot}7/32\,(1/285) = 79{\cdot}5 \text{ tons,}$$

but if the exact fraction be taken it will give 80 tons mean pressure. By formula—

$$W\, v^2/2\, g = W\, h = 0{\cdot}25 \times 20 = 5 \text{ ft.-tons,}$$

or a mean pressure of $\frac{5}{\frac{1}{16}} = 80$ tons as before.

It will be observed that S is taken as the set of the head of the pile, and attention must be directed to one very important point. The distance the head of the pile moves after being struck is the criterion of the resulting force in pounds, but only part of this force is returnable as supporting power; some of it is expended in compressing the pile, and any formula that does not take account of the compression of the pile as well as the penetration of the point can only be approxi-

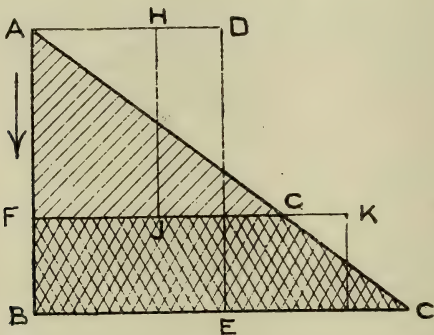

FIG. 3.

mately true. The resistance at the head of the pile begins at zero and terminates at such a pressure that the total movement multiplied by the average pressure equals the foot-pounds energy of the blow. This resistance is shown approximately in Fig. 3, where A B is the total movement of the head of pile at the last blow, and the horizontal ordinates between A B and A C represent the corresponding pressure at each point of the movement, giving an average resistance of A D or B E, which is equivalent to the 80 tons just worked out. Suppose the point moves one-third of the distance moved by the head, then when the head has gone down from A to F the point will begin to move, dividing the triangle of resistances into two areas, the upper one A F G representing the foot-pounds resistance of the pile to

compression with an average of A H or F J, and B E G C the foot-pounds resistance of the point to penetration with an average F K, or B L; but B C represents the ultimate pressure or the resistance in pounds to further penetration, assuming that the resistance varies in direct ratio to the movement. B C is seen to be independent of F B, and therefore the supporting power must be measured by the movement of the head rather than by the movement of the point.

Let W = weight of ram in pounds, and h = height of fall in inches, then B C = 2 W h/A B, or in our notation—

$$U = \frac{24\,W\,h}{S} = 2\,W\,\frac{h}{s} \quad . \quad . \quad . \quad (50)$$

If it should be desired to measure the supporting power by the average resistance of the point to penetration, it will be seen that the area F G C B is $\frac{4}{9}$ of the area A B C, F S being $\frac{1}{3}$ A B, or if s be penetration of the point and S the movement of the head, the proportion of the whole energy exerted which is absorbed in penetration will be—

$$\frac{W\,h}{s} \times \frac{(S-s)^2}{s},$$

and that divided by the depth of penetration will give the average pressure—

$$\frac{W\,h}{s} \times \left(\frac{S-s}{s}\right)^2,$$

or in our notation—

$$U = \frac{12\,W\,h}{s} \cdot \left(\frac{S-s}{s}\right)^2 \quad . \quad . \quad . \quad (51)$$

The test of a pile at Royal Victoria Dock, London, is recorded in *Engineering*, December 29, 1899, p. 826, as follows :—

Pitch pine pile, $12\frac{1}{2} \times 11\frac{1}{2} = 143{\cdot}75$ sq. in., 37 ft. long $= 36{\cdot}8$ cub. ft., at, say, 50 lb. per cub. ft. = 1,840 lb. Weight of ram = 12 cwt. = 2,016 lb.

Driven in peaty ground a distance of 36·1 ft., going $2\frac{1}{4}$ in. to four 8 ft. blows when driving was stopped.

Loaded with rolled joists balanced on head.

Load 10 tons.	No sinking.	Remained thus 24 hours.
„ 20 „	$\frac{1}{16}$ in. „	„ 48 „
„ 30 „	$\frac{1}{16}$ in. „	„ 0 „
„ 37·5 „	$\frac{3}{16}$ in. „	„ 7 days.
„ 56·9 „	$\frac{1}{4}$ in. „	„ 4 weeks.

No further sinking. Load being removed pile rose $\frac{3}{16}$ in.

The rising of the head of pile when the load was removed showed the amount of temporary compression in the timber, and the missing $\frac{1}{16}$ in. may have been further compression held back for a time by the friction of the earth at the sides, and possibly reappearing some time later, a sort of *hysteresis*. The result of the test shows that the pile was practically safe with a load of 60 tons, but like other tests that stop short of completion, it did not show that it would have failed with double the load or more.

The sort of test that is required to prove the accuracy of any formula is to measure the distance moved by the head and by the point during say the last four blows and take the average for each. Cut off a foot from the top of the pile to receive the load on a good surface. Carefully level through from a fixed bench mark to find the level of the top of unloaded piles. Calculate the safe load on the pile and load it up until the head of the pile has sunk to the amount to which it was previously compressed (i.e. previous average movement of head—average movement of point) ; note this load. Then continue loading and note level of pile at each ton addition until the head of the pile has sunk, say, double the previous compression, i.e. previous average movement of head minus average movement of point. Then leave it loaded for 24 hours and note whether it has sunk farther and how much. Remove the load and note the result.

The difficulties in the way are—

(*a*) Finding enough load.
(*b*) Supporting the load on the pile without friction.
(*c*) Measuring the results accurately.
(*d*) Avoiding personal danger throughout the test.

The following example, based upon the author's experience, will serve to illustrate the result of the various formulæ.

A 14 in. by 14 in. creosoted Memel fir pile 30 ft. long weighing $\frac{2}{3}$ ton, driven 12 ft. into soil weighing 90 lb. per cub. ft., having an angle of repose 35 degrees ($\Phi = 0·7$);

represents an average example of timber piling used in jetty work. The last four blows of a 15-cwt. ram with 10-ft. fall would probably drive it 1 in. The safe dead load would then be about 20 tons, the ultimate load being about 60 tons.

The results of the formulæ applied to this example are shown in the table on page 26.

Reinforced Concrete Piles.

The advantages of reinforced concrete piles are so manifest that they need no express recommendation here. The chief physical differences from timber piles, as regards driving and their supporting power, are due to their extra weight and their friable nature. They should be made with slow-setting cement six weeks before driving, but if made with quick-setting cement they must be left eight weeks, because in the latter case the outside hardens first and leaves the interior soft. They should be driven by steam or drop hammer with a 3-ton ram, having a fall of 3 ft., with a steel helmet filled with sawdust, and preferably without a dolly.

In America the use of a water jet is found to greatly facilitate the sinking of concrete piles.

Hollow cylindrical reinforced concrete piles have been used at Southampton, Newcastle-on-Tyne, and Liverpool. They are lighter and cheaper than solid piles and more effective. Those at Brockelbank Dock, Liverpool, were 20 in. diameter. Reinforced concrete piles of circular section are easier to drive than square piles, and as they have no sharp angles are less liable to be damaged by coming into contact with boulders, etc.

As reinforced concrete piles are made horizontally, care must be taken in lifting them; the points of attachment for lifting should not be less than half the length apart, and if lifted with one end on the ground the attachment should be one-third the length from the other end.

The reinforcement rods (about 2½ per cent.) should preferably be hooked at the top and electrically welded together at the bottom. They should be bound helically by, say, ¼-in. wire, 4-in. pitch, carefully secured at the top.

The Nasmyth steam pile-driver quoted by McAlpine (*Min. Proc. Inst. C.E.*, xxvii. p. 277), with a ram of 3 tons and a stroke or fall of 3 ft., gave from 60 to 80 blows per minute. Although the force of the blow was much less the effect was much greater than with ordinary pile-drivers, and the piles (timber) were capable of being driven to a greater depth.

			No.
Weight of dolly ...	Becker	56 lb.	(1)
Size of dolly ...	Adams	6 ft. by 9 in. sq.	(2)
Ratio length of pile to sectional area	$\frac{L}{A}$	$\frac{1}{6\cdot 5}$	(3)
Weight of ram ...	Assumed	15 cwt.	—
,, ,, ...	Dobson	27 ,,	(4)
,, ,, ...	Hurst	15 ,,	(5)
,, ,, ...	Molesworth	—6·54 cwt.	(6)
,, ,, ...	,,	(corrected) 17·6 cwt.	(7)
,, ,, ...	Adams	20 cwt.	(8)
,, ,, ...	Goodrich	$13\frac{1}{3}$,,	(9)
,, ,, ...	Hurtzig	13 ,,	(10)
,, ,, ...	American	22 to 27 cwt.	(11)

Average of (4) to (11) omitting (6) ≑ 18·6 cwt.

	Authority.	Ultimate Load.	Safe Load.	No.
		Tons.	Tons.	
Resistance of pile...	Assumed	60	20	—
,, ,, ...	Sanders	—	45	(12)
,, ,, ...	Merriman	—	60	(13)
,, ,, ...	Berg	—	27	(14)
,, ,, ...	Wellington	72	12	(15)
,, ,, ...	McAlpine	38	$12\frac{1}{2}$	(16)
,, ,, ...	Trautwine	66·4	22	(17)
,, ,, ...	Weisbach (Haswell)	192	19	(18)
,, ,, ...	Ritter	192	19	(19)
,, ,, ...	Baker	211	21	(20)
,, ,, ...	Rankine	223	22	(21)
,, ,, ...	,,	—	$17\frac{1}{2}$–$87\frac{1}{2}$	(22)
,, ,, ...	,,	—	88	(23)
,, ,, ...	,,	—	$17\frac{1}{2}$	(24)
,, ,, ...	,,	$176\frac{1}{2}$–$264\frac{1}{2}$	$17\frac{1}{2}$–$26\frac{1}{2}$	(25)
,, ,, ...	Dutch	190	19	(26)
,, ,, ...	Goodrich	100	—	(27)
,, ,, ...	Haswell	76	17	(28)
,, ,, ...	Hurtzig	37	12	(29)
,, ,, ...	Nystrom	143	24	(30)
,, ,, ...	Rivingtons	—	5–40	(31)
,, ,, ...	Patton	—	9	(32)
,, ,, ...	Howe	—	20	(33)
,, ,, ...	Clark	—	10	(34)
,, ,, ...	Bidder	—	12	(35)
,, ,, ...	French	—	22	(36)
,, ,, ...	Steam driving	—	—	(37)
,, ,, ...	Stickney	—	12	(38)
,, ,, ...	Stickney (steam driving)	—	43	(39)
,, ,, ...	Kreuter (test piles)	—	—	(40)
,, ,, ...	Brereton	93	23	(41)
,, ,, ...	Sankey	81	16	(42)
,, ,, ...	Adams	—	11	(43)
,, ,, ...	Kirkaldy	71	—	(44)
,, ,, ...	Rankine	333	67	(45)
,, ,, ...	Gordon	110	—	(46)
,, ,, ...	Common Rule	—	$6\frac{1}{2}$–14	(47)
,, ,, ...	New York	—	20	(48)
,, ,, ...	Dobson	—	$28\frac{1}{2}$	(49)

Average of 12 omitting (44) = 129 tons and 24 tons.
Average factor of safety = $5\frac{1}{4}$.

This was no doubt due to the earth not having time to settle round and adhere to the piles.

Fig. 4 shows a steam pile-driver at work, and Fig. 5 a section of the ram.

FIG. 4.—Steam pile-driver at work.

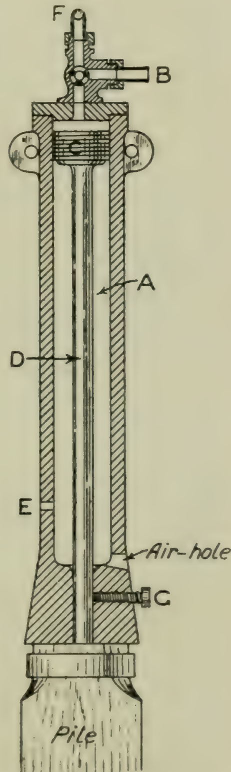

FIG. 5.—Section of steam ram.

It is a debatable question whether reinforced concrete piles are more easily and economically driven to the required depth by a drop hammer or by a steam hammer. The probability is that upon a trial with equal weights of hammer the advantage of the steam pile-driver in the case of wooden piles would be maintained with reinforced concrete piles.

The building laws of most of the cities in the United States

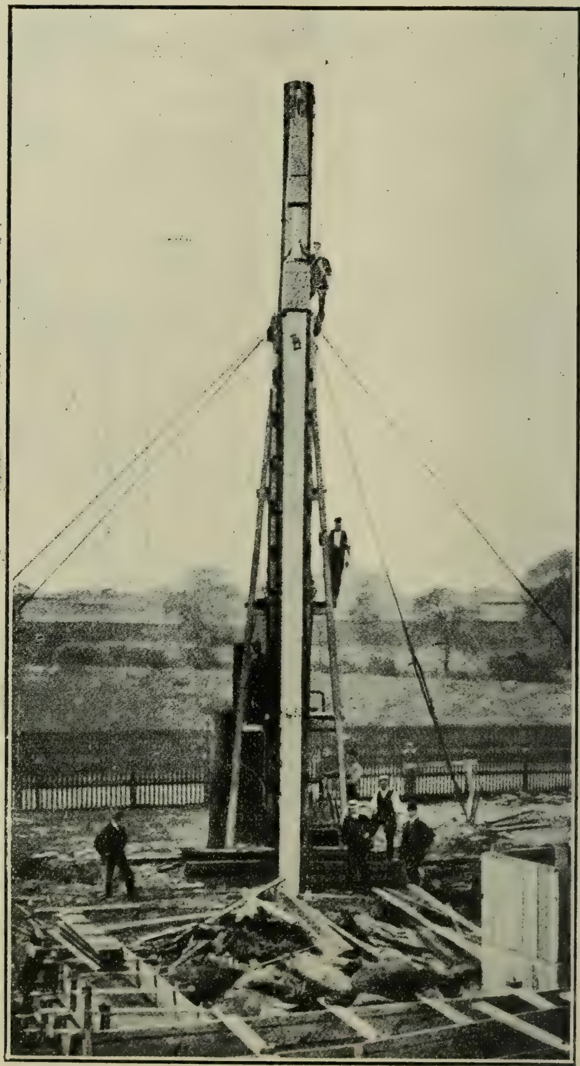

FIG. 6.—Driving reinforced concrete piles, 45 ft. long, in foundations for N.E.R. Co.

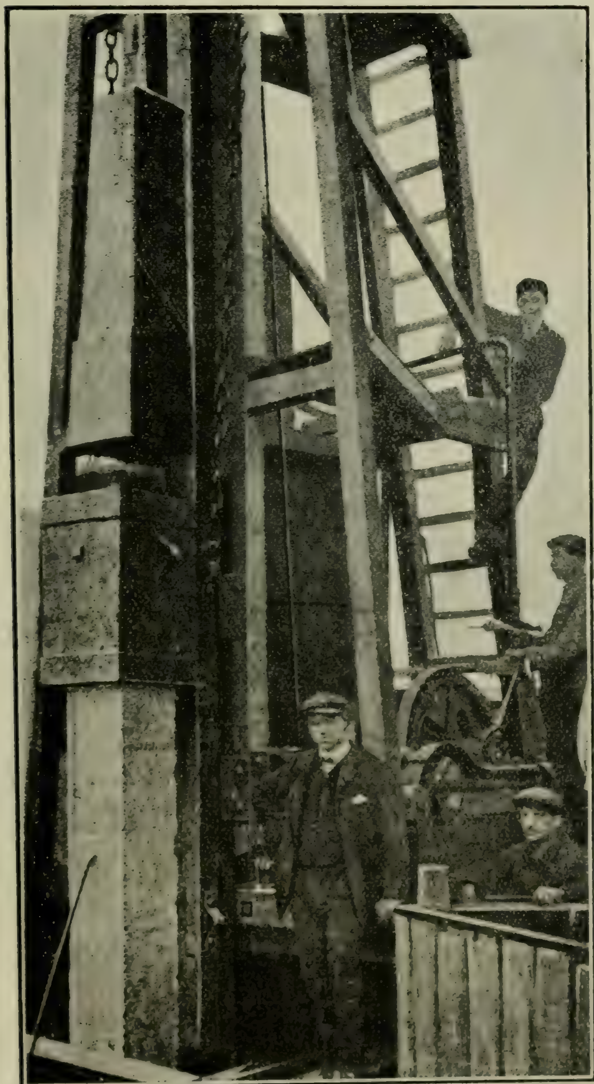

FIG. 7.—Driving reinforced concrete piles, 45 ft. long, 14 in. square, for N.E.R. Co.

allow on concrete piles a safe load from 350 to 500 lb. per sq. in. on the concrete, plus from 6,000 to 7,500 lb. per sq. in. on the vertical reinforcement. This is irrespective of the nature of the soil or depth driven, but they are supposed to be proportioned according to the Wellington or *Engineering News* formula.

Engineering News formula for steam driving—

$$R = \frac{2\,W\,H}{0{\cdot}1 + s} \quad . \quad . \quad . \quad . \quad . \quad . \quad (52)$$

The coefficient 0·1 is the only difference from the drop-hammer formula which has coefficient 1·0. This modification is used in America for concrete piles.

Professor Emil Mörsch quotes the Brix formula as that usually employed for determining the carrying capacity of concrete piles, viz.—

$$p = \frac{h\,Q^2\,g}{2\,e\,(Q+g)^2},$$

where h = fall of ram,
Q = weight of ram,
g = weight of pile,
e = average penetration of pile under last few blows,
p = double the safe allowable load for pile.

This translated will be—

$$R = \frac{W^2\,W_p}{4\,(W+W_p)^2} \cdot \frac{h}{s} \quad . \quad . \quad . \quad (53)$$

This formula is unitally complete.

Note that if $W = W_p$ the Brix formula would become—

$$R = \frac{W\,h}{16\,s}.$$

This should be compared with Major Sanders's formula No. (12). In the above case the safety factor is 4, so that $U = \frac{W\,h}{4\,s}$. This should be compared with formula (27).

Taking the weight of reinforced concrete pile as 3·6 times the weight of fir pile = 2·4 tons; the weight of ram as 3 tons,

and fall 3 ft., the other conditions being as in the example of the timber pile, we shall have—

$$R = \frac{3 \times 3^2 \times 2{\cdot}4 \times 3}{0{\cdot}25\,(3 \times 2{\cdot}4)^2} = 15 \text{ tons safe load,}$$

while by the Wellington formula—

$$R = \frac{2 \times 3 \times 3}{0{\cdot}25 + 0{\cdot}1} = 51{\cdot}43 \text{ tons safe load,}$$

showing the same discrepancy as in the case of the formulæ for timber piles.

It is a commonly received notion that a reinforced concrete pile will safely carry double the load that a timber pile of similar dimensions would do, but the writer has not found any data from which such a conclusion could be absolutely derived. He has not found a single case of a test load which has caused a concrete pile to sink farther. There are many accounts of loads put on piles which did not cause any sinking, but these have no practical value for checking the formulæ.

Mr. Charles Gow, in a paper read before the Boston Society of Civil Engineers, considered 30 tons the safe approximate load on concrete piles, but the size of the piles was not stated.

In the Dittman Factory, Cincinnati, Chicago, one concrete pile was considered equivalent to two or more wooden piles.

The "Raymond" concrete piles are not reinforced, but are made by first driving with a solid steel core a thin conical steel shell, which excludes sand and water, and is filled with concrete after the core is withdrawn. For the Friede Globe Tower at Coney Island, 700 ft. high, where the soil consisted entirely of fine set sand, the concrete piles were about 8 in. diameter at the point and 20 in. at the top, driven to a depth of about 30 ft. They were calculated to have a bearing capacity of 300 tons each, but are loaded with only 57 tons each.

In the case of the 14-in. square reinforced concrete piles of the wharf at Lower Pootung, Shanghai (*Min. Proc. Inst. C.E.*, 1911–12, clxxxviii. p. 80), driven in river silt with a crust of stiff sandy clay, and designed to carry a load of 11 tons each, it was estimated that the skin friction was 5 cwt. per superficial foot, exclusive of their own weight,

with a final set of 1 in. per blow from a steam pile-driver with a falling weight of $3\frac{1}{2}$ tons dropping 6 to 8 in. upon a short timber dolly and no cap. The additional dead weight of cylinder, etc., resting on the pile was $1\frac{1}{2}$ tons. In one pier the four piles sank rapidly under the dead weight of the hammer and cylinder to the required depth without any driving, and had to be checked by holding up the weight when they reached that depth, but after the lapse of a month these four piles were tested with a load of 45 tons, under which they settled $1\frac{3}{4}$ in. in a fortnight, coming to rest at the end of that time. This example shows the special value of skin friction in the case of concrete piles.

Screw Piles.

Screw piles of reinforced concrete have been patented by Mr. Vernon Inkpen, of Portsmouth, and they would probably be found very useful in a peaty or wet sand foundation.

Rankine's "Useful Rules and Tables" say, "The load supported by a screw pile in practice ranges from 3 to 7 times the weight of earth which lies directly above the screw blade." This seems to be a very peculiar way to measure the supporting power, and appears to be based upon the resistance to the "spewing-up" effect of the downward pressure by the screw blade, and not on the direct resistance of the soil below the blade to compression. The fact is that the soils vary so much between the extremes of plastic clay and firm gravel that whatever rules may be laid down, personal judgment must come in as the most important factor, whatever kind of pile or method of driving may be employed.

In conclusion, the author desires to express his thanks to Mr. E. Fiander Etchells for kindly reading over the manuscript, elaborating the notes relating to Standard Notation, and making other suggestions.

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The Concrete Institute:

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The Care of Ancient Monuments

By C. R. PEERS, Sec.S.A.,

Chief Inspector of Ancient Monuments and Historic Buildings to H.M. Office of Works and Public Buildings.

It may well seem to members of this Institute that the problems which confront them to-day have little enough to do with the matters on which I propose to speak, and indeed the limitations of material under which the old builders worked confined their ideas within much narrower bounds than ours. But the art of construction is a very ancient art, and many of the results of modern science have been anticipated by rule of thumb centuries ago, to be forgotten and rediscovered in different surroundings. A knowledge of such things is essential in dealing with ancient structures, where a mind in sympathy with the methods and ideals of the past is the only sure guide to a right treatment. It will be obvious at first sight that this will limit our choice of expedients; we must rule out treatments which are convenient and advantageous enough in new works, but incompatible with the old. I hasten to add that I do not for a moment suggest that the results of modern science are inapplicable to ancient buildings, or that we should use no processes which were not known to their builders. Far from it; our claim to be a generation which values its inheritance of history must rest on our employment of all the means which are at our disposal, for the preservation of that inheritance. But they must be used in the right way, and from this spring the limitations which must be observed. An ancient monument, speaking generally, has

three precious qualities: its history, its beauty, and its educational value; in attempting to prolong its existence we must not obscure or destroy these qualities. If something must be sacrificed to preserve the rest, the distinction between essentials and non-essentials must be clearly defined, or perhaps it would be more accurate to say that the relative importance of parts which are all by the nature of the case important must be apprehended. The monuments under the charge of the Department of Ancient Monuments and Historic Buildings range from earthworks and megalithic monuments to seventeenth century houses, and demand an equally wide range of treatment. They present in infinite variety examples of the ills to which antiquity is subject, but their dilapidation, when not due to intentional damage, may be said to arise from two main sources, damp and structural weakness; the accumulated shortcomings of nature and man.

Few people are insensible to the attractions of age in a building, which by no means consist only in the surface colour and texture which nothing but time can give. To those who can read it, an old building offers a more intimate and authentic record of its makers than almost any other relic of past times. Matters of common use, the small details of life, every day occurrences which no one, then or now, considered or considers worthy of record, survive for us there, and from such small human things we may often gain a truer historical sense and understanding of our own position in the world's development than from all the written records of statecraft, war and commerce.

We are accustomed to hear comparisons drawn between the work of former ages and our own, not to our own advantage. This is by no means always fair. There has been good and bad building in all ages, and in the course of nature more of the bad buildings have perished than of the good, and in consequence the achievement of any period which has left an appreciable number of works is liable to be judged on too favourable a ground. The Romans were great engineers, and the mortar of such of their buildings as survive in Great Britain cannot be improved on and hardly equalled at the present day. But in the fifteen centuries which have passed since they left Britain all the inferior Roman buildings have perished, and even in those that are left there is certainly no uniform standard of merit.

The walls of the Roman town of Caerwent, on the Welsh border, are faced with levelled and bedded masonry which was built one course at a time on both faces of the wall, and set in hard and well mixed mortar. The core of the wall, between the facing stones, was then put in; it consisted of dry stones of irregular shapes set roughly on edge. A bed of coarse mortar was laid over the dry stones, bringing the surface to the level of the top of the facing courses, but not filling all the voids in the core. Then another line of facing course was built, and so on. The mortar in which the facings are set is much better than that of the core, and quite weather proof. But the core, except for its great thickness, is not strong enough to resist a failure of foundations, and if once exposed to the weather will let in the wet and soon become disintegrated.

At Cardiff the process of building was similar, but in several ways better. Two to four facing courses were built at a time; the space between them was then filled with pebbles and odd stones, and the whole consolidated by pouring in a liquid grout, which filled up all the voids and made a thoroughly strong construction.

At Richborough, in Kent, the wall core is a concrete, mixed and thrown in between the facing courses and levelled up. The walls are solid throughout and of such strength that the cutting of holes ten feet and more in width right through the thickness of the wall from side to side has in no way weakened the masonry above.

The Roman tradition of building with two faces and a core was continued in the middle ages, but often with none of the care and thoroughness which was necessary for its success. In the 11th Century, at any rate, the core in many instances was little more than earth and building rubbish packed in between wrought stone faces, these latter in small stones with shallow beds. Such walls would stand no great weight and were also particularly sensitive to any foundation movement or lateral stress, having no natural strength.

In a small building, where stresses are neither great nor complex, a weather proof wall face protecting a weak core will often serve well enough for the time, but the ruin or reconstruction of many of our mediæval buildings has followed the adoption of such a principle. Walls were pointed in tolerable lime mortar, but built in nothing but clay, and as long as the pointing was able to keep the

weather out, they were able to do the work for which they had been designed. But if, through any settlement or stress, a fracture developed, the masonry had no power of resistance, but fell away and became fit for nothing but pulling down, for lack of sound walling to which to bond a repair. It will easily be seen that it is almost impossible to strengthen such a wall so as to prolong its existence appreciably, without destroying its character, considering that its character is the very source of its weakness.

So much for the evil arising from the degradation of a tradition; but the dangers inherent in an imperfect scheme of construction, incidental to the growth of a style, are equally difficult to deal with. An overloaded arch or pier, an ill-calculated thrust, seem to demand for their complete cure so much substitution of new work for old, or such disfiguring ties and supports, that the balance of gain over loss to an ancient building draws perilously near to nothing.

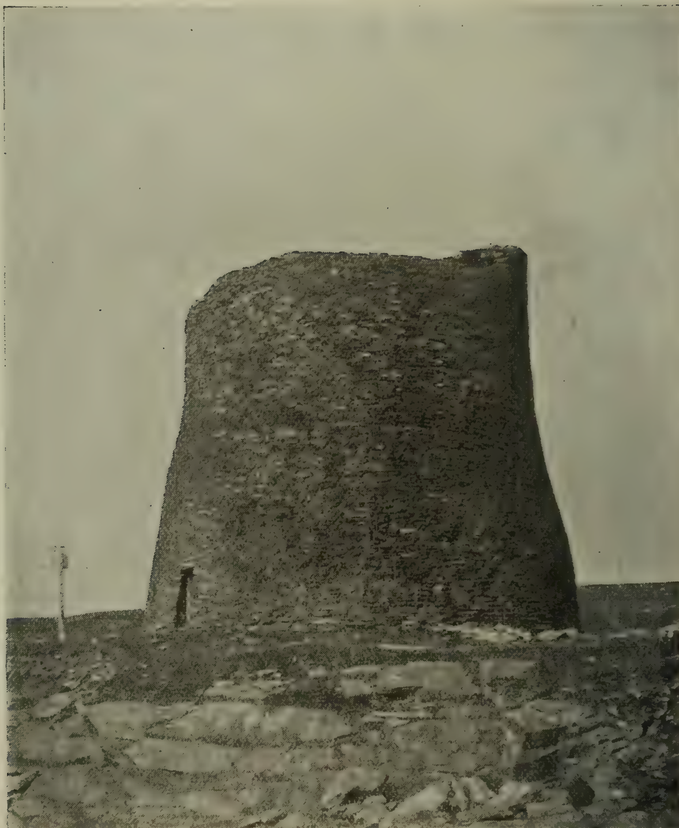
A third evil, for which at present no adequate remedy has been found, is the decay of stone. This is a particularly important matter, as the loss of the surface of an ancient building, though not necessarily affecting its stability, is disastrous for its history and appearance. The causes of stone decay are various, but damp is an almost constant factor. By its agency acids which attack the structure of a stone are carried into its pores, and while a dry surface remains perfect, a ledge on which water can stand, a moulding from which it can hang, or a face down which it commonly runs, will all begin to decay. The cementing material of the stone is attacked and its particles become loose and fall away; and the evil, once started, is progressive and not to be stopped, as has been often attempted, by the application of a weather proof solution to the surface.

A series of experiments, having for their object the discovery of a really effective treatment, has been in progress for some time at Edinburgh, instituted by the Commissioners of Works; but though certain phenomena have been definitely established, it cannot be said that any general principle of treatment has yet been laid down. The difficulty lies not so much in getting a preservative solution to sink into the stone as in preventing it being drawn out to the surface again in the process of crystallisation and evaporation. I will not take up your time to-night with

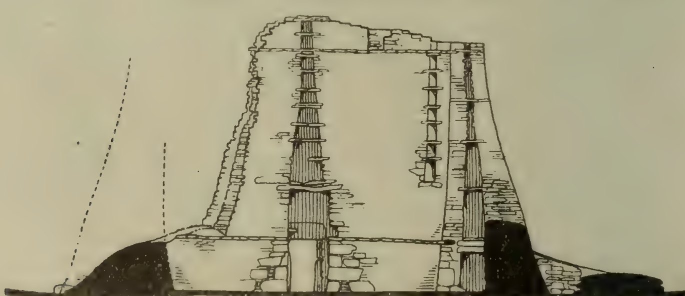
further reference to this question, vitally important though it is to the work of my department; but it will easily be understood that if it is ever successfully answered, one of the greatest of the difficulties under which we now labour will have been completely removed.

I shall now propose to invite your attention to some typical examples of the repair of ancient buildings, carried out during the last five or six years, and will begin with a prehistoric monument, the lower broch of Glenelg, in the extreme north-west corner of Inverness-shire, at the head of the Sound of Sleat.

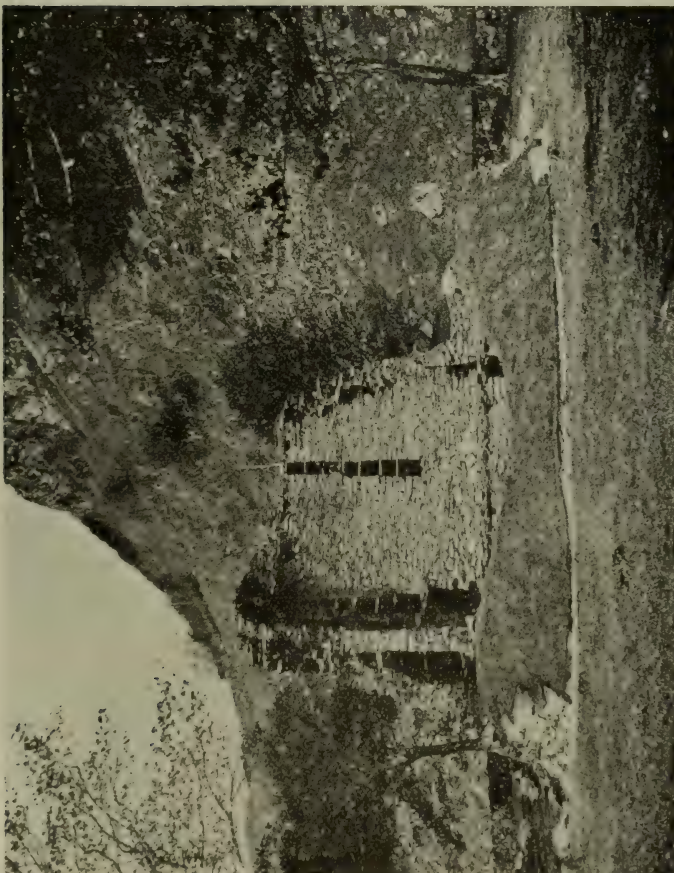
Brochs are peculiar to Scotland, and from the evidence of the objects found in them, may have been first built as early as the 2nd Century B.C., but seem to have continued in use for many centuries afterwards. The last reference to the occupation of a broch belongs to the year 1155, the broch in question being that of Mousa in Shetland, which happens to be by far the best preserved of all those that remain. A broch is a circular building like a hollow tower, with an internal diameter varying between 20 and 40 feet and walls averaging 12 to 15 feet in thickness. Chambers, passages and stairs are formed in the thickness of the walls, which rise in the case of Mousa to a height of 45 feet, and have a very marked batter on the outer face. Except for a single doorway at the ground level there are no openings on the outer face of the walls, but on the inner face tall vertical openings, bonded across at intervals, give light to the wall passages and stairs, which in the upper part of the broch divide the wall into an inner and outer thickness. The whole is built dry, without mortar of any sort, and relies for its strength on the thickness of its walls and the tie-stones which bond them together. When such a building begins to fail, it is obvious that any patching with new stone or pointing the joints is out of the question, and would entirely alter its character. In the case of Glenelg only a small part of the circle of the tower remains, and the wall, 34 feet high from the ground, showed a tendency to fall inwards, the unmortared stones being able to slide one on the other. A temporary strengthening with wooden shores had been put up some ten years before, but this had ceased to be effective and was of course a disfigurement. The remedy was to consolidate the walls in a way which did not appear on the surface, and this was accomplished by filling in the joints on each face six inches deep with



Mousa Broch, Mousa, Shetland.



The Lower Broch of Glenelg: Section and internal elevation.



The Lower Broch of Glenelg.

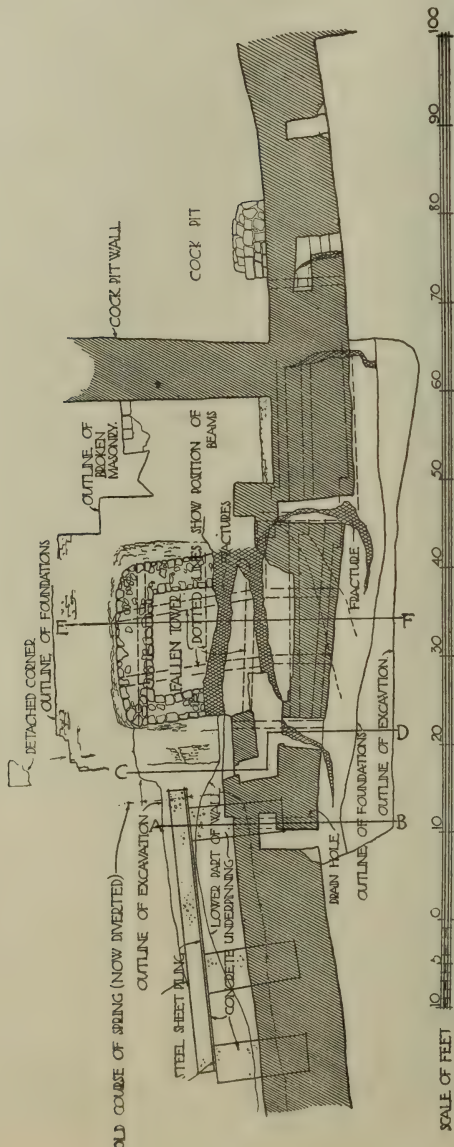
clay, beginning from the base of the wall and working upwards, and filling the spaces left in the middle of the walls between the clay with cement grout. This having set hard the clay was removed and the walls washed down, leaving the cement-set core invisible from either side, and the original appearance of the masonry unaltered. The lower parts of the walls needed no treatment of this kind, and are as they were first built. What was left of the rest of the circumference, where only a few feet of walling remained, was capped with turf to pin down the uppermost stones.

In grouting the wall it was, of course, not necessary to use a machine, the grout being poured by hand into the hollow of the wall.

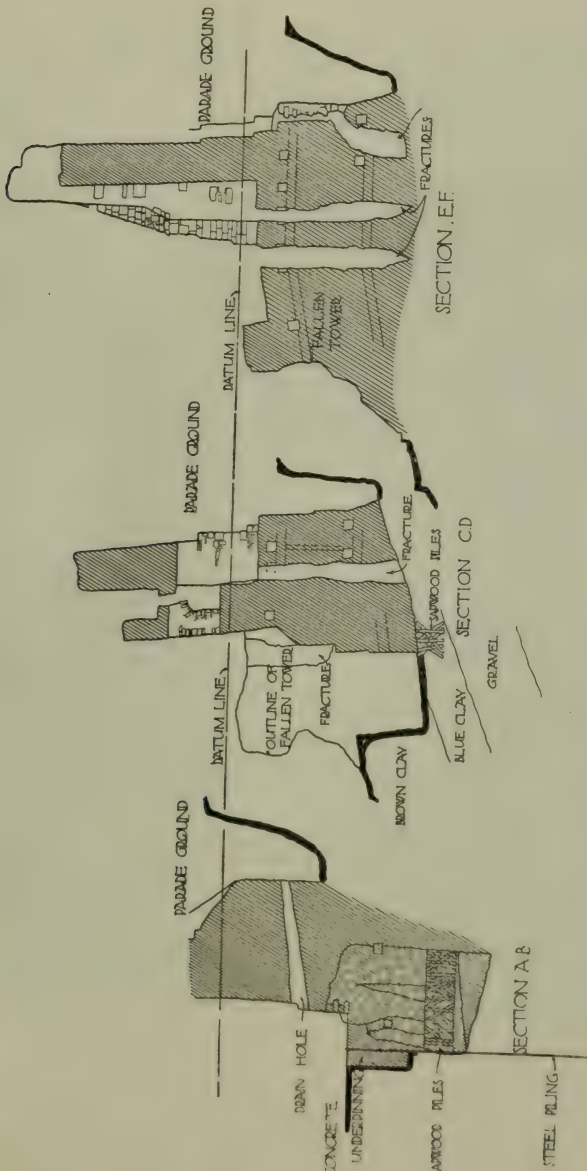
I will now take the repair of the east curtain wall of Richmond Castle, Yorkshire. This is part of the oldest masonry in the castle, having probably been built within ten years of the Norman Conquest. The site of the castle is a spur of rock overlooking the river Swale, and the levels fall quickly on the west, south and east. The walls are founded on the rock, except on the east side, where a section showed that below the surface soil there is a 9ft. layer of brown clay resting immediately on five feet of blue clay. Below this is a bed of gravel, 8 feet thick, and again below the gravel more brown clay, the rock being only reached at about 30 feet. These beds are set with a considerable downward slope towards the east, and form a natural drainage line for the castle area, so that they are constantly wet, and the upper layer of brown clay tends to slide on the blue clay beneath it.

For reasons of defence it was necessary for the Norman builders to carry the east wall of the castle along this line, in spite of the unsatisfactory foundation, of which they were well aware. To counteract the tendency to slide they cut through the brown clay a trench 15 feet wide, carried down to the blue clay into which a number of wooden piles were driven. These were about 3ft. 6in. long by 3 to 5 inches in diameter, and were evidently cut on the spot, being of various woods such as birch, holly, etc. Where the blue clay was thin they entered the gravel below, but this was evidently by chance and there was no attempt to reach the gravel in every case. The object of the piles was to consolidate the foundation, the upper part of the blue clay being very soft from the constant draining of

water over it; in some places the whole bed of the foundation trench was piled, in others only a narrow band of piles ran along its outer side, the rest of the clay being presumably considered hard enough for the work it had to do. The piles were set at all angles, some vertically and others diagonally; from their tops to the original level of the ground, some 10 feet, the wall was built in the trench, of water worn boulders in puddled clay thrown in between built faces; from the ground level upwards the wall was built in lime mortar, with a more regularly laid core. The whole was strengthened by a framework of wooden beams embedded in the masonry as a bond. The beams were laid in tiers about 7ft. apart vertically; longitudinal beams averaging 30ft. in length and 9 inches square were set about 3ft. 9in. apart, centre to centre, and notched over transverse beams of similar scantling. The end of each longitudinal beam was lapped over the next beam for about 5ft., and perhaps bolted to it, and the tiers were connected by vertical posts 9in. by 6in. Such a construction has a very early origin, being an obvious method of tying together any body of small masonry, and was used in early dynasty brick walls in Egypt several thousand years before our era. It is also a natural way of strengthening an earthwork, and Cæsar tells us that the defences of the Gaulish fortress of Alesia were made of wooden framings of the sort filled in with earth and turf. But it will not last in a damp situation like this at Richmond, and had in fact completely rotted away, leaving empty chases which had become a source of weakness and not of strength. How soon the wall began to fail it is hard to say, but an extant survey of 1538, when the castle was rather more than three and a half centuries old, mentions that part of it was then overhanging. The greatest stress came on the line of natural drainage, as might be expected, and probably within a century of the date of the survey mentioned the tower which here projected from the wall heeled over and tore away, causing great longitudinal rents in the masonry, the fractures following the line of the chases for the beams. The wall itself was similarly dislocated, and began to move downhill at the same time, leaning forward dangerously as it did so. At its extreme point of movement the wall has moved four feet from its original line, and overhangs as much as 28 inches. The tower has moved six feet downhill, and but for some old underpinning would



Richmond Castle: Plan of East Curtain and Fallen Tower.

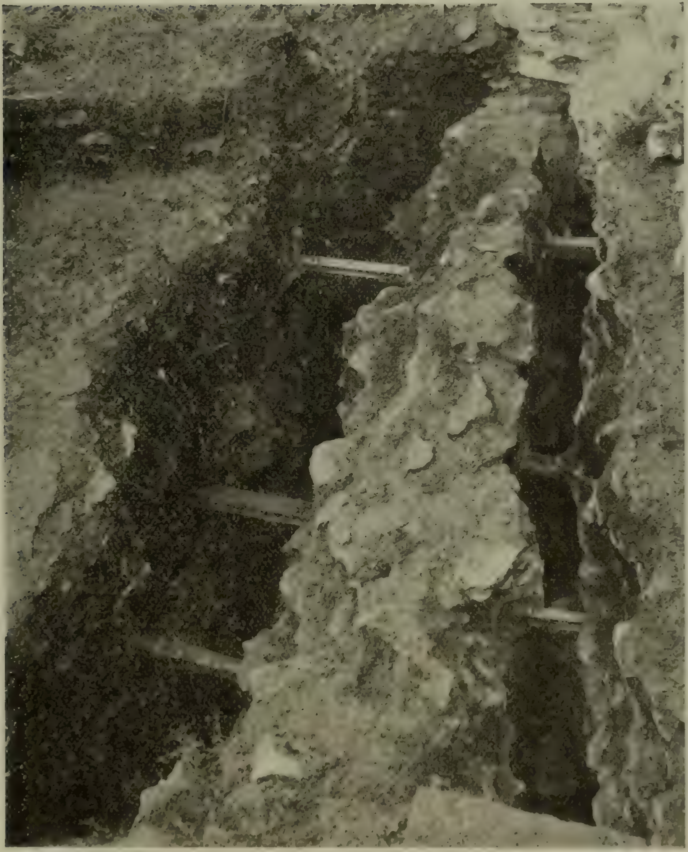


Richmond Castle: Plan of East Curtain and Fallen Tower.

have fallen into the castle ditch. The only repair that had been made, until the last few years, took the form of cutting back the overhanging outer face of the wall to a vertical line, a process as futile as it was destructive, and the strain on the shattered masonry had been greatly increased by a levelling up of the area of the castle for purposes of military drill, the effect of which was to pile some ten additional feet of earth against the inner face of the wall.

The repair by the Ancient Monuments Department began in 1912. The origin of the trouble being the wetness of the foundation, the first step was to collect the water into a channel and prevent it spreading along the line of the wall. This carried away the greater part, leaving enough to keep the clay subsoil damp and so to prevent any shrinkage, but the more serious question of anchoring down the foundation of the wall remained to be dealt with. To underpin the wall was not enough without some form of buttressing, which if only for appearance sake could not be allowed to show above the ground level. Mr. F. Baines, the architect in charge, therefore decided to put in a line of sheet piling in front of the weakest part of the wall, at 3ft. 6in. from its outer face. The piles were 30 foot steel interlocking piles, and this length was found to be more than enough to reach the underlying rock. These being driven, the clay and pebble foundations of the wall, with the layer of blue clay beneath, were removed in 4ft. lengths and replaced by concrete bedded on the gravel and pinned up to the underside of the old lime-mortar-built masonry, a height of 14 feet; this foundation was carried back for about three-quarters of the thickness of the wall, and brought out to the line of the piles; at the same time a bank of concrete, 7ft. deep by 2ft. thick, was set against the outer face of the piles to counteract their tendency to spring at the top.

To attempt to rebuild the overhanging wall tops was out of the question, as the history and authenticity of the masonry would have been entirely destroyed in the process, and it was also clear that the wall was massive enough, when properly strengthened, to resist the strain of the leaning stonework. The rents in the base of the wall, and the hollow chases left by the rotting of the bond timbers, were therefore filled in solidly with concrete, while the dislocated masonry of the upper parts was secured by bonding stones at intervals, the voids being filled in with loose stone which



Richmond Castle: Cracks between East Curtain and Fallen Tower.

was afterwards solidified by grouting with a machine at a pressure of 30lbs.

I spoke at the beginning of this paper of the need to sacrifice something in an ancient building in order to preserve the building. What is the loss in this case? Some 30 feet in length of the old foundation, and the wooden piles beneath, have been destroyed, and with them all that an untouched piece of 11th Century construction can teach us or our successors. Full measured and photographic records have been preserved, it is true, but these are after all at one remove from actuality and cannot answer all the questions which a future age may wish to put. The gain is that the whole wall above the foundation is preserved indefinitely, we hope, in as genuine a condition as may be, for the pleasure and instruction of posterity. If nothing had been done most of us who are here to-night would probably have lived to see its final ruin.

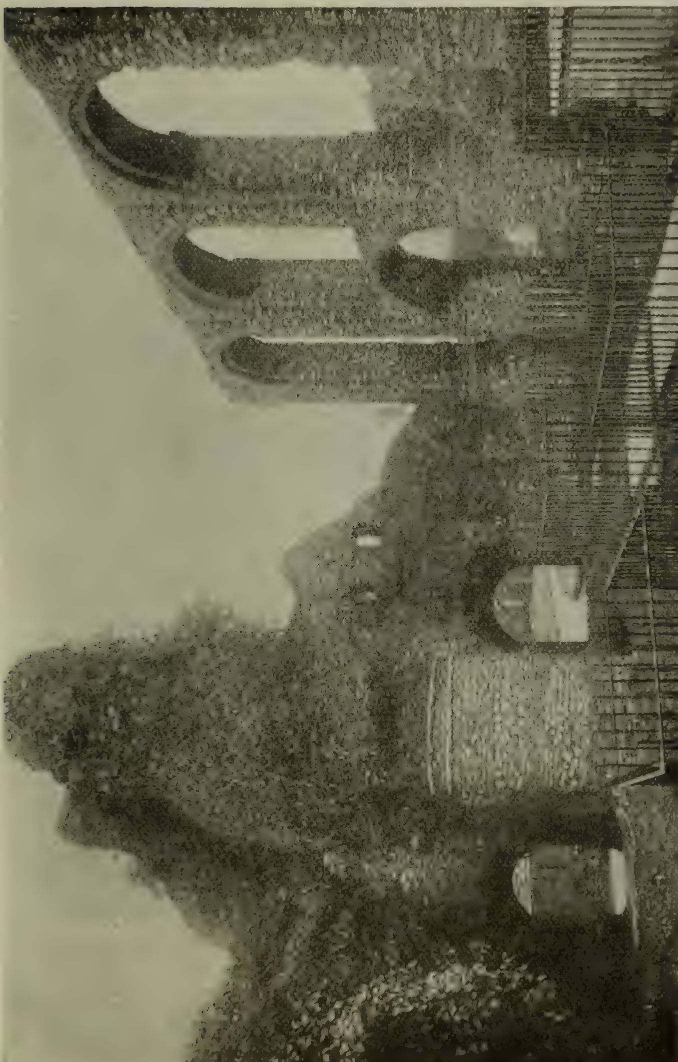
In addition to the loss of a part of its fabric, there are minor losses, inevitable in the repair of any old building. Centuries of neglect have allowed the rain to soak into its walls and plants and bushes to grow upon them, thrusting their roots deep into the masonry and pushing the stones apart. The mortar has gradually yielded to damp and frost and crumbled to dust, or has fallen out, leaving the empty joints to act as channels for rain water. All decayed mortar and earth must be raked out of the joints to a depth sometimes of a foot to 18 inches; the roots must be pulled out, the voids filled and the facing masonry pointed to keep the wall waterproof. Very little of the old pointing, if it survives at all, will be sound enough to do its work, and as a result nearly the whole of the joints in the walls will be new, and until their newness has worn off the general effect will be far less picturesque than before. The surface of the mortar joint is therefore of great importance, and it may be well here to describe the practice of the department in raking out and repointing. All pointing, except on horizontal surfaces and wall tops, is in lime mortar, hydraulic limes being used, as for example, in England, blue lias lime from Leicester or elsewhere; in Scotland, Arden lime; in Wales, Aberthaw lime. The sand is to be as coarse and sharp as possible, and in order to bring it to the surface of the joint it is the practice to spray the joints with water before the mortar is set, in order to wash away the particles of lime and leave the coarse grit exposed. If the joint is

more than three inches deep, it is backed with cement mortar up to three inches from the face to avoid the drawbacks of the very slow setting of a large body of lime mortar. Cement pointing is to be avoided because of its colour and its hard and inelastic nature, and therefore greater tendency to crack away from the sides of the joint when set. On wall tops, however, and horizontal ledges it must generally be used, as lime mortar in such places is liable to get soaked with rain and in consequence to break up in frosty weather. If cement is used with a very coarse pebbly grit, breaking up its surface as much as possible, its ugly grey-white colour is less noticeable.

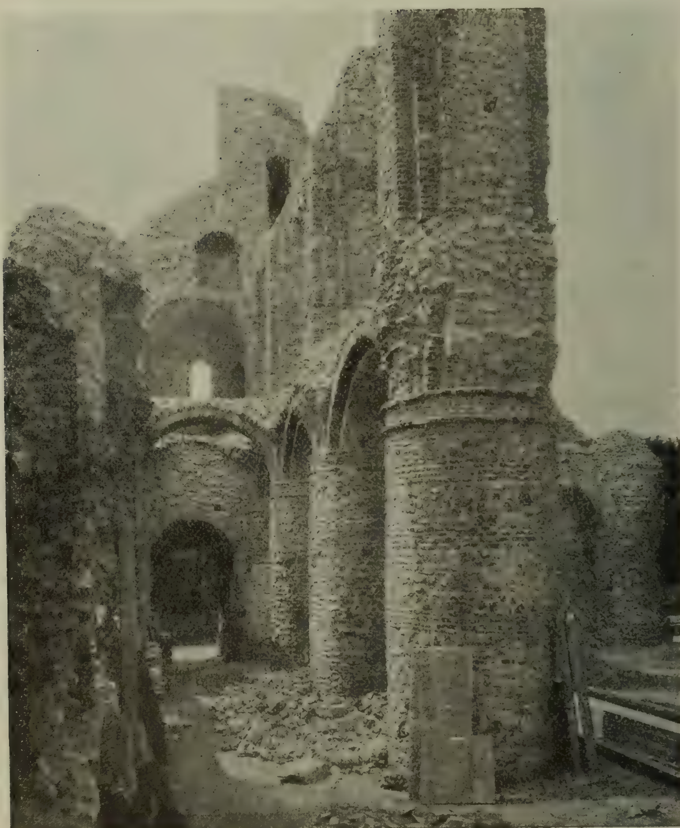
To give an instance where practically complete re-pointing and resetting were necessary, I will take the ruins of the Priory Church of St. Botolph, Colchester. Essex is, of course, not a stone country, and buildings not of brick or timber are usually built of flint rubble, plastered. St. Botolph's, which dates from the 12th Century, was the church of a house of Augustinian Canons, the nave of which continued in use after the suppression, but was much damaged at the siege of Colchester in 1648 and has since remained in ruins. It is built of flint rubble with dressings of Roman brick, all originally plastered, and only in doors and windows is stone employed. Such a building is very susceptible to the climate, when it has once lost its coat of plaster, and when it was handed over to the Department of Ancient Monuments it was in a deplorable condition, besides being smothered in ivy and other parasitical plants. The walls had in many places been stripped of their outer coat of flint masonry, leaving a crumbling core exposed, in which the mortar was little better than dust and the masonry could be pulled in pieces by the hand. Mortar surfaces in such a building are large, and when the walls had been cleaned and cleared of vegetation and sound mortar substituted for what was decayed, the effect was undoubtedly somewhat raw. But time will bring the whole into harmony again, with the additional advantage that the masonry is now sound and strong.

An important piece of work which was fortunately well advanced when the war broke out is the repair of Jedburgh Abbey Church. The danger here, and it was very considerable, arose from the building of the 12th Century tower in the "core and facing" construction to which I have referred above.

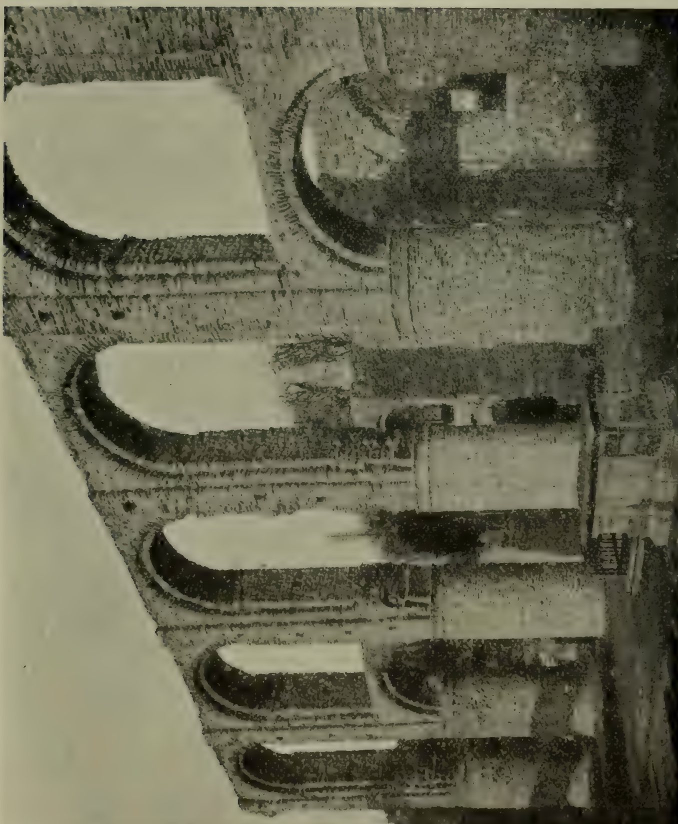
St. Botolph's Priory, Colchester: S.W. Corner of Nave, before repair.



St. Botolph's Priory, Colchester: Internal View of Nave, before repair, looking West.

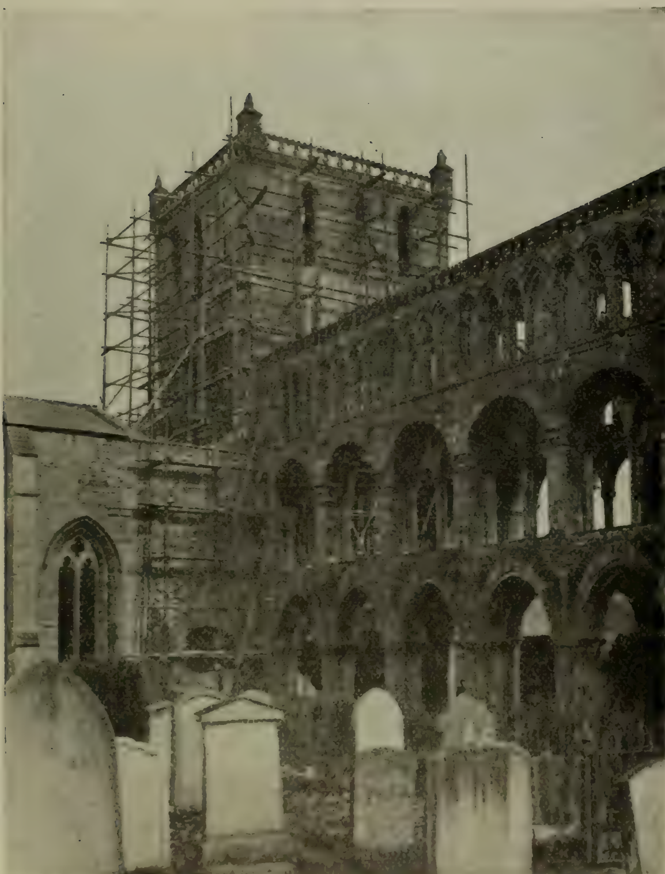


St. Botolph's Priory, Colchester: South Arcade of Nave, after repair, looking West.

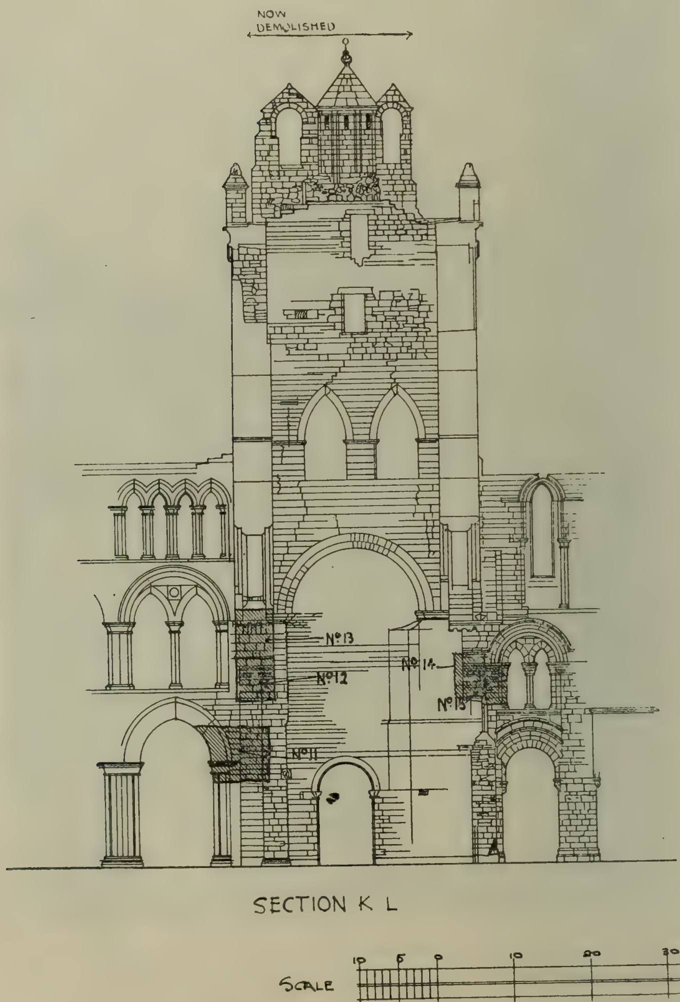


St. Botolph's Priory, Colchester: North Arcade of Nave, View after repair.

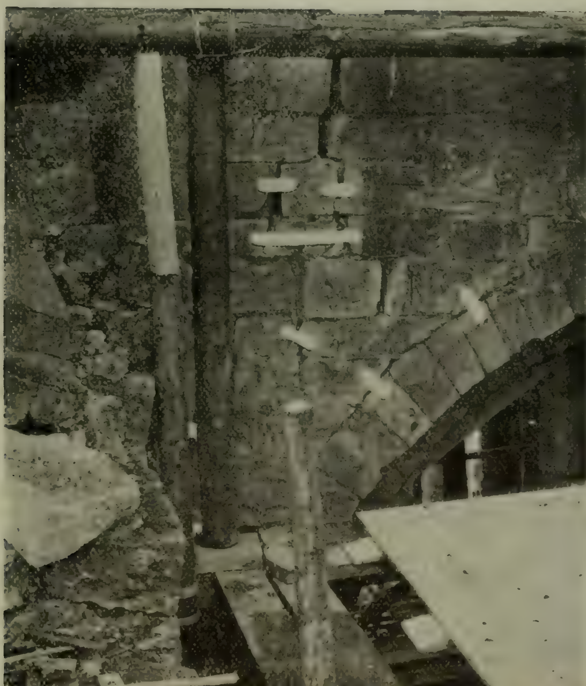
Jedburgh Abbey was founded by David I. of Scotland in 1118, when he was as yet only Prince of Cumbria, but was refounded by him in 1147, and it is to this latter date, or soon after, that the earliest parts of the present church belong. The church is cruciform with a tower over the crossing, and was originally designed with a short eastern arm of two bays and probably an apse, transepts with eastern apsidal chapels, and a nave of nine bays. The first work, belonging to the middle of the 12th Century, is now to be seen in the western half of the eastern arm, the transepts, and the lower parts of the tower. The tower has undergone many alterations, its south side and the greater part of its east and west walls having been rebuilt in the 15th Century, while the upper stages of the tower belong to the early years of the 16th Century. Though of no great height, the added masonry is massive and heavy and was at one time more so, before the fall of the pointed stone barrel vault which covered in the upper stage. The northern crossing arch with the north-east and north-west piers, and the masonry for some feet above the crown of the arch, being part of the original work, have failed under the weight of the later masonry, and have only been preserved to our time by the blocking of the arch as high as its springing with a solid stone wall. The old wall core is of very poor quality, a mass of unbedded rubble in weak lime mortar, and the crushing weight of the tower has come on the stone facings, which are only a few inches on bed, and as is so often the case with old masonry, taper inwards from their squared outer faces. The results may best be realised from the photographs I show; the crushing was evident over the whole north side of the tower, with a definite tendency to a settlement north-westward. The cause of the failure was clearly the weakness of the core; if it had been able to do the work of carrying the superstructure—a work which, it must in justice be added, it was never intended to do—the shallow facing stones would have served their purpose very well. The alternatives were to take down and rebuild the tower with stronger material, or to replace the old core with something better. There could be no doubt as to which was the appropriate treatment, when tested by the rule that the greatest possible amount of old work must be preserved, and the process adopted was to remove the old core piecemeal from the foundations upwards and replace it with solid concrete.



Jedburgh Abbey, May, 1914: Tower from N.W.



Jedburgh Abbey: Section of Tower, looking N.



Jedburgh Abbey, May, 1914: This photograph shows fractures on N.E. spandrel of N. Arch of Tower. A few of the fractures are hidden by surface plastering and the poles. Five tell-tales on the spandrel were cracked.

Jedburgh Abbey, May, 1914: This photograph shows fractures on S. face of N.E. pier where the rubble casing was removed. This portion of the pier is in a crushed and very shattered condition. The fractures from the capital to the spring of the arch above are merely filled up with surface grouting.

This was naturally a risky and difficult work, but the scheme devised by Mr. Baines, the architect in charge of Ancient Monuments, was carried through successfully and the new concrete core has been carried up to meet the 15th Century masonry above.

Before anything could be done it was, of course, necessary to shore up the tower, the north, east and west arches being centred, and the north wall, which as already said was in the most unstable condition, steadied by shores from the north transept walls. A system of dead shores, strutted or braced together to secure entire rigidity, carried the needling by which the weight of the upper parts of the tower was borne during the process of re-coring. The north-east and north-west angles of the tower were held up by a triangular needling of rolled steel joists carried on the dead shores. The joists were set in threes; two sides of the triangle, those going at right angles through the walls, measuring 15 inches by 6 inches, while the third side, composed of joists 24 inches by $7\frac{1}{2}$ inches, went diagonally through the angle of the tower.

The process of re-coring was as follows:—A small section of facing stones at the base of one pier was carefully removed to a height and width just sufficient to allow access to the core within, and for greater security against possible movement of the face, screwjacks were inserted and tightened up to steel plates on the underside of the stones at the top of the opening and on the upper face of the stones at the base of the opening. With the core the danger of a fall of material was naturally to be feared directly any part of it had been hollowed out and removed. Steel plates 4 inches wide and $\frac{1}{4}$ inch thick were therefore provided, with one pointed end, which could be driven into the wall core and temporarily supported at the outer end, being tightened up with folding wedges as required. A start being made on one face of the pier, the core was removed over about half its area, in heights of a few feet at a time, and replaced by concrete, which was put down in layers, so planned and stepped that each additional layer should be overlapped and bonded to subsequent layers, avoiding any danger of a straight joint in the new filling of the piers. Steel rods were also used to tie the blocks together, being embedded in the concrete as it was laid in. The old core was removed up to the backs of the ashlar facing, the joints of which were thoroughly

cleaned out and tamped in cement mortar at the same time. One side of the pier having been treated in this way, the other side was then taken in hand from below upwards, each piece of new core being filled in carefully to the line of that already in place, and the ashlar facing reset as the work went upwards. So the work was carried up into the haunches of the crossing arches, where the remains of the 12th Century triforium passage were found built up in the heart of the wall. The 15th Century builders had filled them in for strength, and it was reluctantly decided that it was unsafe to open them out again; they were therefore built up solidly in concrete. During the carrying out of this work an elaborate system of telltales and levels was in use, so that the slightest movement of the tower could have been kept under observation, but only on one occasion, when a particularly violent storm of wind occurred, was any movement noticed, and the whole operation, it is hoped, may be said to have been carried through successfully. It will now be possible to free the tower from the unsightly blocking walls which have long hidden its northern side, and from the wooden shoring which for a good many years has so much injured the effect of this beautiful church.

It would be possible to multiply instances, but I hope that those which I have described will sufficiently show the application of the principles laid down at the beginning of this paper. In such a meeting as this it is not necessary to dwell on the absolute need of the best materials and the most careful workmanship; the members of your Institute can learn nothing from me on that head. But of all structures which come under our care to-day, a ruin needs the most painstaking treatment; being exposed on all sides to wind and weather, and weakened by the loss of much of its masonry, it exists in a condition for which it was never designed, and against which all our experience is not yet fully adequate to protect it.

It goes without saying that the work to which the nation's energies are now devoted leaves little time for such things as I have laid before you; they must suffer an eclipse till the day of destruction has passed and that of rebuilding has begun. But of all the lessons that the war has taught us, the greatest, perhaps, is that we should be well prepared for our future tasks, whatever they may be, and the making good of the losses inflicted on the historical monuments of the world will not be the least of such tasks.

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The Re-modelling of an old Graving Dock at Southampton.

By R. N. SINCLAIR, M.C.I.

FLATTENING FLOOR.

The Graving Dock known as No. 3, which forms the subject of this paper, was built in 1853 from the design of the late Mr. Alfred Giles, Past President Inst.C.E., and was lengthened to its present dimensions in 1882. It is shown on the plan and section, Figs. 1 and 2. It has an inside length of 520 feet and a width at entrance of 80 feet. The keel blocks are placed at a level of 24 feet below H.W.O.S.T., the blocks themselves being 3ft. 3in. high. The Dock is closed by a pair of wrought iron gates which, when opened, lie back in recesses, and when closed shut against a stone cill which is raised about 2ft. 6in. above the rest of the floor. From the cross-section it will be seen that the walls are 15 feet thick at the base, and that they are reduced by means of three sets of altars to a thickness of 5 feet at the top. The floor has the shape of an inverted arch, and formerly contained five small altars or steps on each side. The walls are constructed of brickwork in lime mortar. The floor consists of the same material for a thickness of 4 feet, and below that of a similar thickness of lime concrete. The altars are capped with Portland roach stone, and the coping of the walls and the hollow quoins, against which the gates rest, are of the same material. The cill is of Bramley Fall sandstone, which was

considered to be a very fine building stone at the time the dock was built. It is curious that granite, although much used in the older docks at Southampton, is hardly seen in this particular dry dock. There are no precise records of the nature of the subsoil on which the dock is placed, but there is no doubt that it consists either of a weak sandy clay or of a fine running sand, both of which strata prevail in this neighbourhood. It is probable that fine sand highly charged with water occurs under most of the floor. This conjecture is confirmed by the fact that at the south end of the dock there are two vertical holes through the brickwork of the floor through which water is constantly coming up.

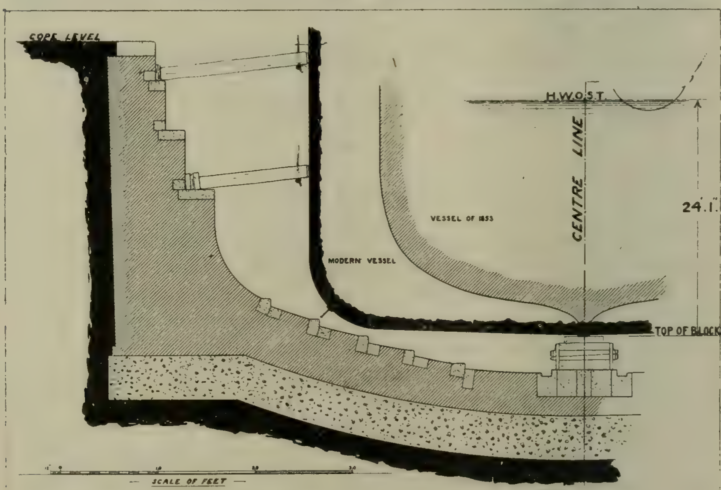
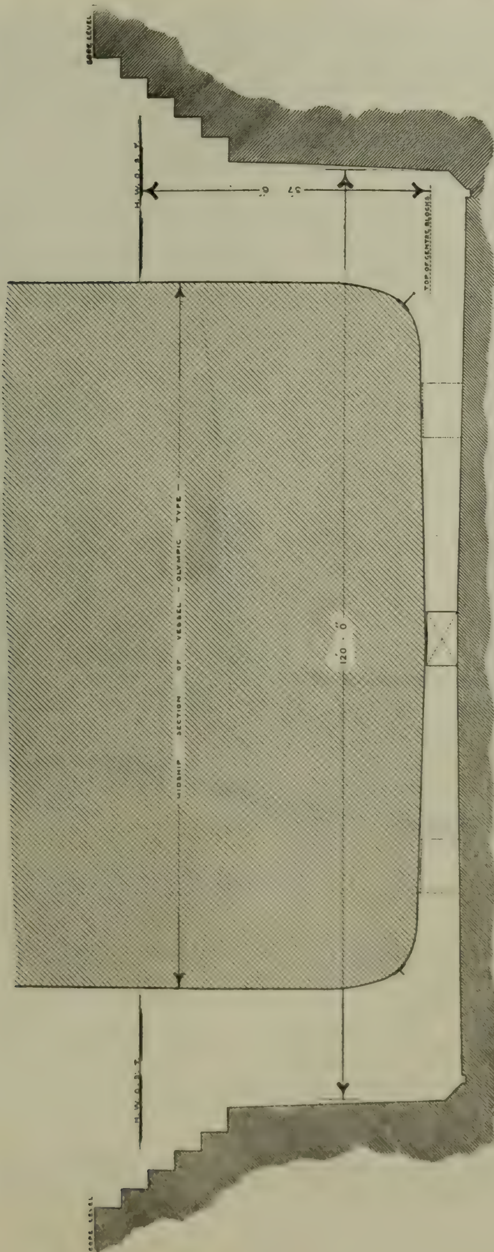


FIG. 2. CROSS SECTION BEFORE REMODELLING.

These suggest that the builders were unable to block out the subsoil water which found its way through the concrete base of the floor, and that they were therefore obliged to provide these two outlets to allow it to pass into the dock.

During the period intervening between 1853, when this dock was built, and the present time, a great change has taken place in the design of ships. Fig. 2 shows the midship section of two vessels which illustrate this change. It will be noticed that not only are the modern vessels larger, but they are practically flat bottomed, and that the rise of floor and the easy curves at the bilge of the older



TRANSVERSE SECTION

FIG. 3. A MODERN DOCK.

ships are now almost done away with. A corresponding change has had to be made in the shape of dry docks to accommodate these modern vessels, and Fig. 3 shows the cross-section of a dock designed to take the very newest vessels which have yet been built.

No. 3 Graving Dock at Southampton was, however, designed to take the old-fashioned ship, and when the more flat bottomed vessels were introduced it was found almost impossible to get them in and out of the dock unless the keel blocks were raised. This meant that such ships could only pass in at very high tides and that deep draft vessels

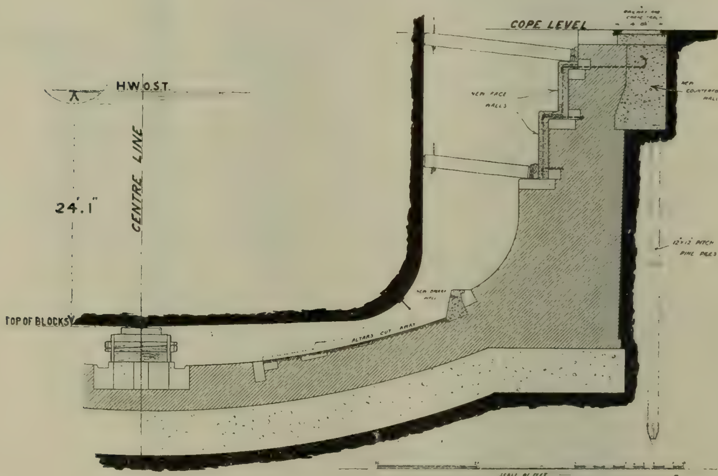


FIG. 4. CROSS SECTION AFTER REMODELLING.

could not go in at all. Again, even if they could be got in, the bilges were so close to the floor that it was impossible to get round them to carry out repairs, or even to paint them properly. The usefulness of this dock was therefore seriously reduced, and in order to make it once more available for modern ships it was decided to flatten the floor by cutting away the haunches of the inverted arch, as shown in cross-section, Fig. 4.

Before doing this, however, it was necessary to ascertain how the strength of the whole of the structure would be

affected by the proposed alteration. It must be borne in mind that the heaviest stresses on a dry dock, built on such soil as occurs here, are produced not by the weight of the ship but by the upward pressure of the sub-soil water on the under side of the floor when the dock is empty. This pressure induces crushing stresses in the inverted arch of the floor, and also tends to lift the whole dock bodily as if it were a ship. Thus, the effect of cutting away the haunches of the floor would be twofold. In the first place the thickness of the arch would be reduced and the arch itself consequently weakened; and in the second place the weight of the floor would be reduced, and therefore the tendency of the whole structure to lift would be increased.

As regards the reduction in the strength of the arch, it will be seen that the only way to compensate for this would be to cut out and rebuild what is left in some stronger material. This no doubt could be done, but it would probably be an expensive and risky operation, owing to the pressure of the sub-soil water, which would be difficult to hold in check. It was decided, therefore, to cut away the floor only to the limited extent shown on Fig. 4, as it was considered that this could be done with safety.

To assist in arriving at this conclusion an experiment was made to find out the actual upward pressure of the sub-soil water beneath the floor. Two small holes were drilled through the whole thickness of the floor, one at each end of the dock to the sand beneath. These holes were partly lined with a wrought iron tube firmly fixed in position by grouting. To the tubes were screwed a pressure gauge which stood up above the floor and registered the pressure of the water which ran up inside the bore hole when the sand was reached. Readings were taken every hour for 24 hours, and it was found that the pressure stood steadily at 7lbs. per square inch at the south end and 5lbs. per square inch at the north end. It was curious that the bore hole nearest the entrance to the dock gave the lower pressure, probably because the sub-soil was more watertight at this point. It was also curious that the pressure did not seem to be affected by the rise and fall of the tide. The larger pressure, 7lbs. to the square inch, corresponds to a soakage level of 10 feet below H.W.O.S.T. This was assumed, therefore, to be the level from which pressure head should be measured. Fig. 5 shows a stress diagram in which half the floor arch, as cut away, is considered to

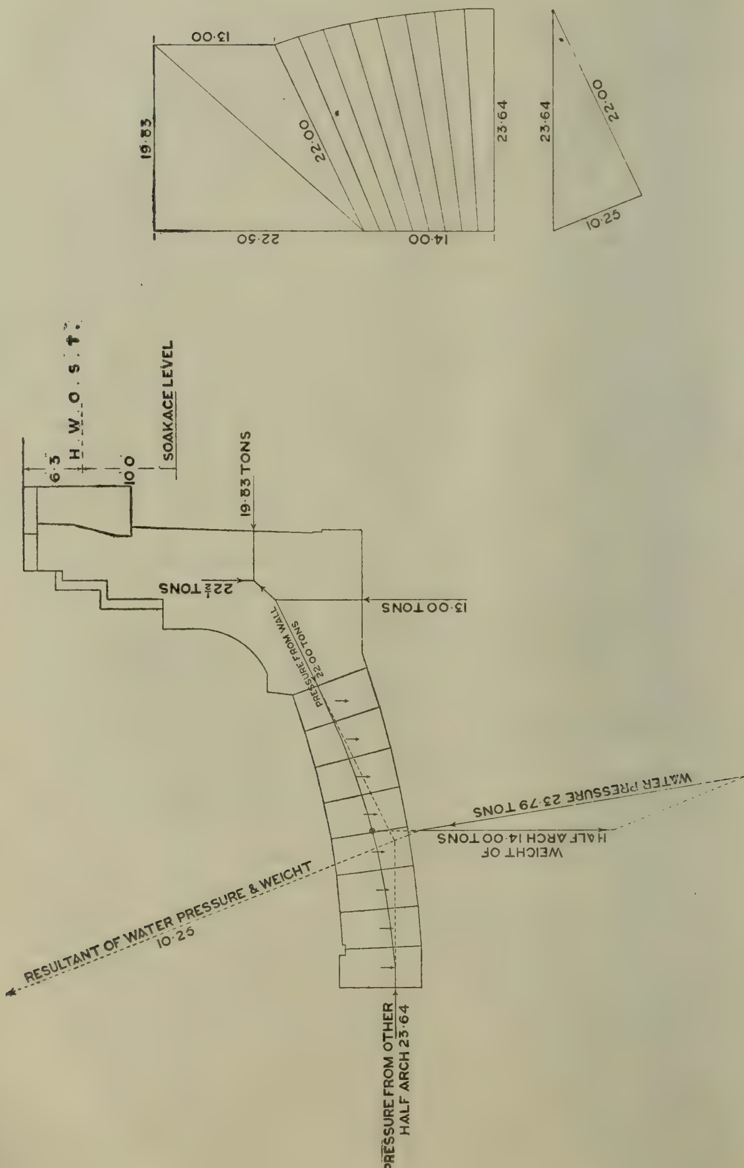


Fig. 5. Stress Diagram.

be in equilibrium under four loads, viz., its own weight, the upward pressure of the sub-soil water as found above, the horizontal pressure from the other half of the arch, and the inclined pressure from the wall. The centres of the two latter pressures being assumed to act at one-third points, the maximum pressure on the masonry is found to be $5\frac{1}{2}$ tons per square foot. It was considered that even the lime concrete, locked in as it is by firm ground and good brickwork, ought to be able to bear this pressure safely.

As regards the loss of weight on the floor, it was of course an easy matter to compensate for this, and the method adopted is shown on Fig. 4. The principal compensation took the form of a counterfort wall built at the back of the existing wall. Additional weight was also provided by thickening out the two altar walls. It will be noticed that the compensating weight was greater than that of the floor cut away. This was done to make sure that there should be absolutely no danger of an upward movement of the walls and floor. It has been noticed that if the dock is kept filled with water for some days the soakage level of the ground gradually rises. Assuming that the soakage level is 10 feet below high water, there is now an excess of weight over flotation of $4\frac{1}{2}$ tons per foot run of dock, but should the soakage level rise, this excess might be reduced to danger point. The compensation walls were therefore made heavy enough to guard against such a contingency.

It will be seen from the cross-section, Fig. 4, that the cutting away extended from a point 12 feet from the centre of the dock to the floor altar at the foot of the wall. In order to make a convenient pathway for workmen in the dock, this floor altar was widened to 2 feet by means of a small concrete wall. The new counterfort wall at the back of the old wall was made of 6 to 1 Portland cement gravel concrete, 4ft. to 5ft. 6in. wide and 8ft. deep. It rested partly on timber piles driven at intervals of 10 feet, and partly on a special benching cut in the back of the wall. The top of it was tied on to the existing wall by $1\frac{1}{4}$in. steel ties 5 feet apart; these ties passed through the wall and were hooked into the new altar walls. Whilst carrying out these improvements it was decided to lay a railway track on each side of the graving dock to accommodate a travelling steam crane. This, indeed, was the principal reason for introducing the piles, as the ground below the counterfort

was quite unsuited to bear heavy loads. The new altar walls were made of 4 to 1 Portland cement gravel concrete and were 12in. thick. To prevent them being knocked away by the rough usage which these walls invariably suffer, they were strengthened with No. 15 expanded steel, 6in. mesh, $\frac{1}{8}$in. × $\frac{1}{8}$in. strands. They were also secured to the old walls by means of two lines of Lewis ties, and also by the long ties which have already been mentioned. It will be noticed that these walls serve the double purpose of compensating for the weight cut away and also of widening the top altar which was formerly much too narrow for safe working.

The works were commenced in January, 1914, but owing to the necessity for keeping the graving dock in constant use they have been frequently interrupted. To commence with, the piles at the back of the wall were driven; and it was laid down that the penetration should not exceed 1/20th inch with a 30 cwt. ram falling 5 feet. The piles had to be driven 30 to 45 feet below ground surface before this test could be obtained. A timbered trench was then excavated for the counterfort wall, and the benching was cut out of the back of the old wall with hammers and points. At the same time the holes for the steel ties of the new face wall were drilled by hand, and the ties themselves were fixed and grouted. The concrete for the counterfort was next placed and the new rails for the cranes laid. This part of the work was completed in October, 1914, nine months after its commencement.

A long interval elapsed owing to the dock being in constant use, and then in February, 1916, a start was made with the cutting away of the floor, the dock being handed over to the engineers for that purpose. The cutting away was all performed by labourers with hammers and points. The brickwork came away fairly easily, but the stone floor altars were more difficult to cut. One line of stone had to be uprooted bodily, and the groove thus formed was filled up with 4 to 1 concrete. The debris from the cutting was all barrowed to either side of the dock, where it was lifted out by a travelling 3-ton crane into railway waggons, which were hauled away to a spoil heap. Meanwhile the new floor altar walls and upper altar walls were built behind shuttering, details of which are shown in Fig. 6. The concrete was mixed on the quay close by the dock by means of a $\frac{3}{4}$ cubic yard electrically-driven Ransome mixer. The

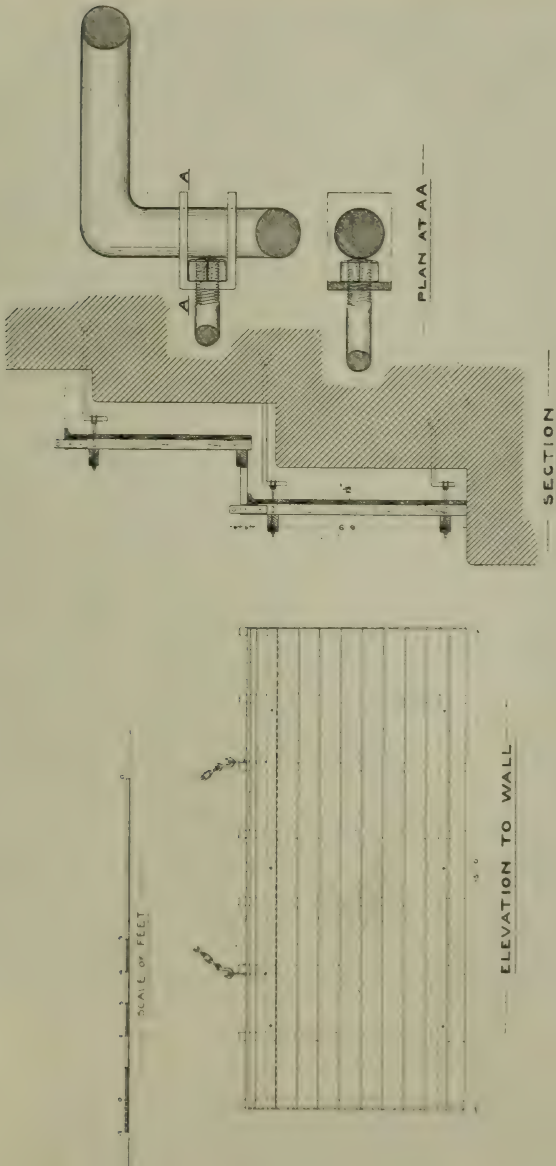


Fig. 6. Details of Shutters.

materials were brought up to the mixer in railway trucks, and the concrete was lowered down in skips to the various walls by the 3-ton crane. The work was completed in September, 1916, the dock being closed to traffic for a period of seven months. It must be explained, however, that this period would have been shorter but for the necessity of carrying out other work, which will be described later.

A curious incident occurred when the cutting away of the floor was nearing completion. The dock had been filled with water and very slowly pumped out, the emptying occupying some days. When finally dried out it was found that the floor on the east side had risen in the form of a large blister about 60 feet long and 15 feet wide, and about 5 inches high in the worst place. To prevent further movement it was loaded with heavy stones, and at the same time a vertical hole was drilled about three feet into the brickwork in the centre of the blister. On reaching this depth water at once rushed out of the hole and continued flowing for some days, after which it flowed intermittently. It was found that the beds of the brickwork had separated where this blister occurred, and at first it was feared that it might be due to the horizontal pressure induced in the arch by the sub-soil water. It was observed, however, that the blister tended to settle down again under the weight of the stones and that the wall showed no sign of movement. It was concluded, therefore, that the trouble was simply due to water under pressure getting in between the layers of brickwork and forcing them apart. To remedy this a number of vertical, or rather radial holes, were drilled about three feet into the brickwork at intervals of about five feet. Into these holes were inserted 1in. steel ties upset at both ends, and each was carefully grouted in with neat cement. This arrangement of ties was carried out over an area of floor much larger than the blister. Most of the holes were dry, or at least free from running water when grouted, but through two of them water flowed so freely that they could not be grouted, and it still continues to flow. No attempt was made to stop it back, and no further trouble has occurred.

REPAIRS TO GATES AND CILL.

As mentioned above, the dock was closed with a pair of wrought iron gates. These are probably over 60 years old, and although they had been repaired from time to time they

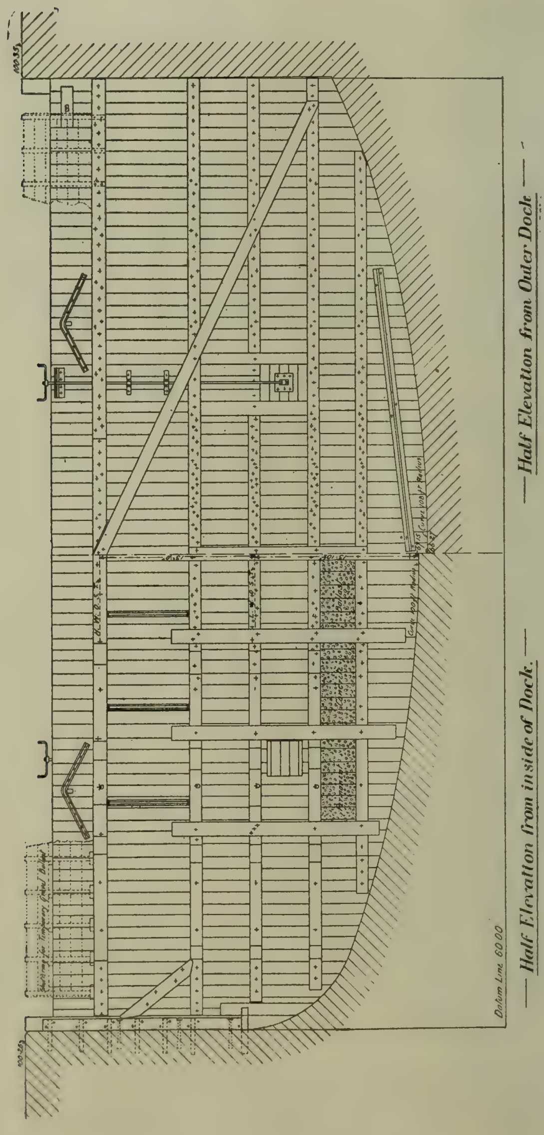
had got into a bad state, so much so that about the time that war broke out it had been intended to renew them. It was found, however, that no manufacturer would guarantee delivery of new gates, even at war prices, and it was therefore decided to repair the old gates as far as possible so that they might remain in use until the war was over.

While this was under consideration an accident occurred to the masonry of the cill, against which the gates rested. The nature of the accident will be described later, but here it may be said that it necessitated the immediate erection of a dam at the extreme north end of the dock so as to admit of the entrance being dried out, and repairs to the cill, and incidentally to the gates, to be put in hand.

TEMPORARY DAM.

The type of dam adopted consisted of a single floating gate made of timber and strutted with the same material. This type was adopted in preference to any sort of pile dam, because it was cheap, and moreover it could be easily removed and used again when the time came for renewing the dock gates.

The dam is shown on Fig. 7 and 8. The gate proper consists of a sheeting of upright pitch pine 13in. timbers bolted to five lines of horizontal walings. Four of the latter consist of three whole timbers inside the dam and one outside. These walings were connected on the inside of the gate by six vertical timbers and on the outside by two diagonal timbers, to prevent racking. The second waling from the top is also tied to the sheeting by strong "Z" irons made of old rails. The joints between the sheeting timbers were all caulked with oakum. Two sluices 3ft. × 3ft. were cut in the sheeting below low water level, these being closed by timber penstocks. The gate was, of course, cut as accurately as possible to the shape of the entrance of the dock, and all round the edge of its face was fastened a line of padding formed of canvas tubing filled with oakum. This served to ensure a watertight joint between the gate and the stop against which it rested. This stop consisted of a step in the brick floor, which fortunately had been constructed at the mouth of the dock by its builders. For some reason or other, however, they did not continue this stop up the walls, and consequently a stop had to be constructed on each side. This was formed of a 13in. timber,


Half Elevation from Outer Dock
Half Elevation from inside of Dock.
Datum Line 60.00

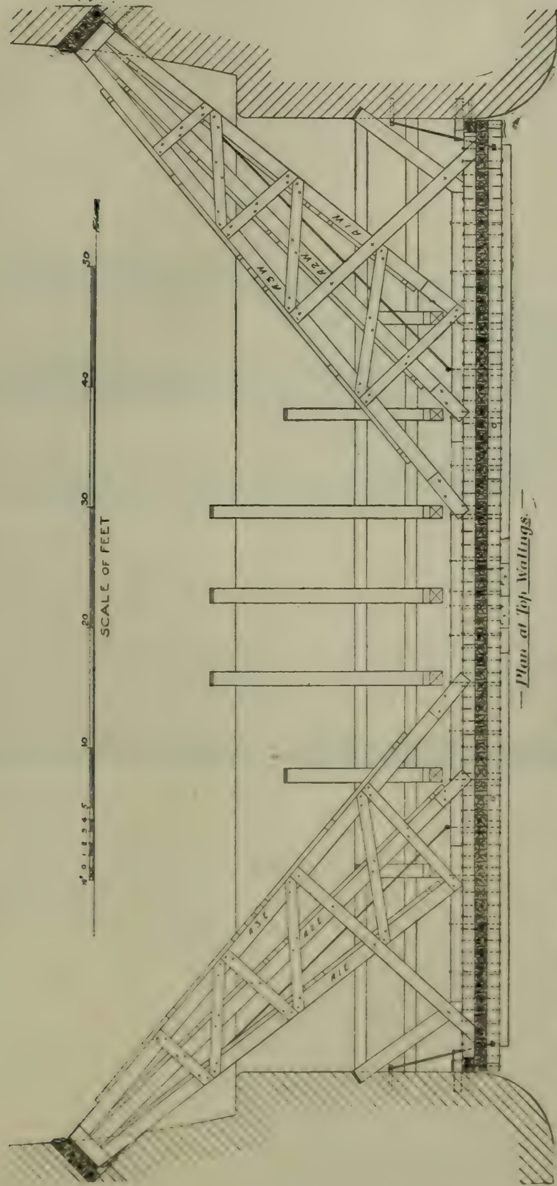


Fig. 7. L. & S.W.R. Southampton Docks: Dry Dock No. 3: Temporary Dam at Entrance as Constructed.

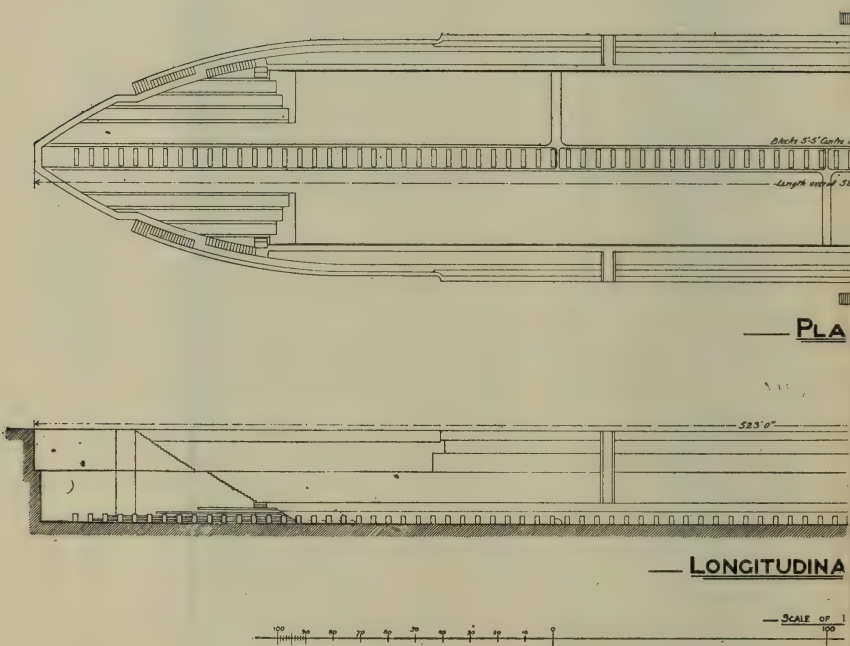
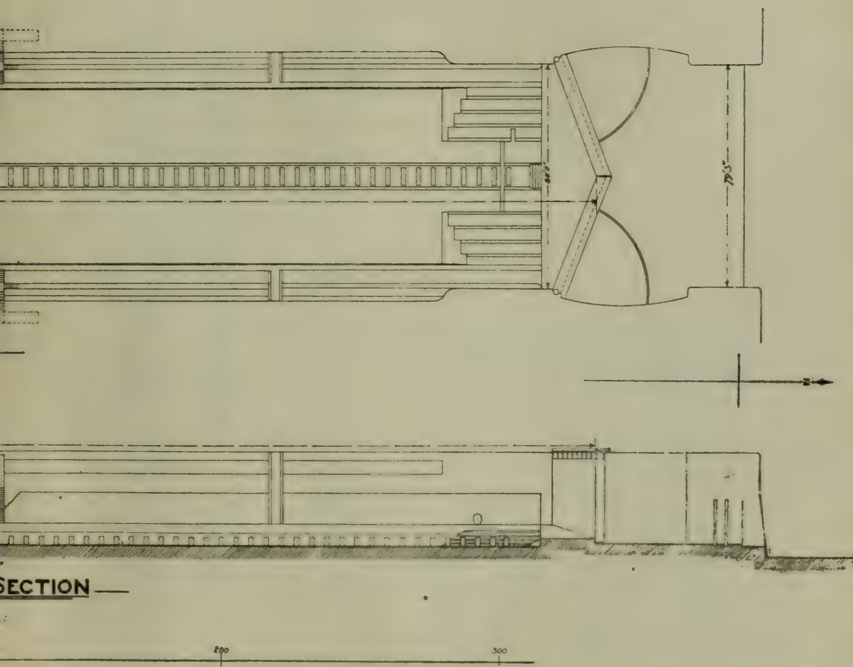


FIG. 1. L. & S.W.R. SOUTHAM



...on Docks: Dry Dock No. 3.

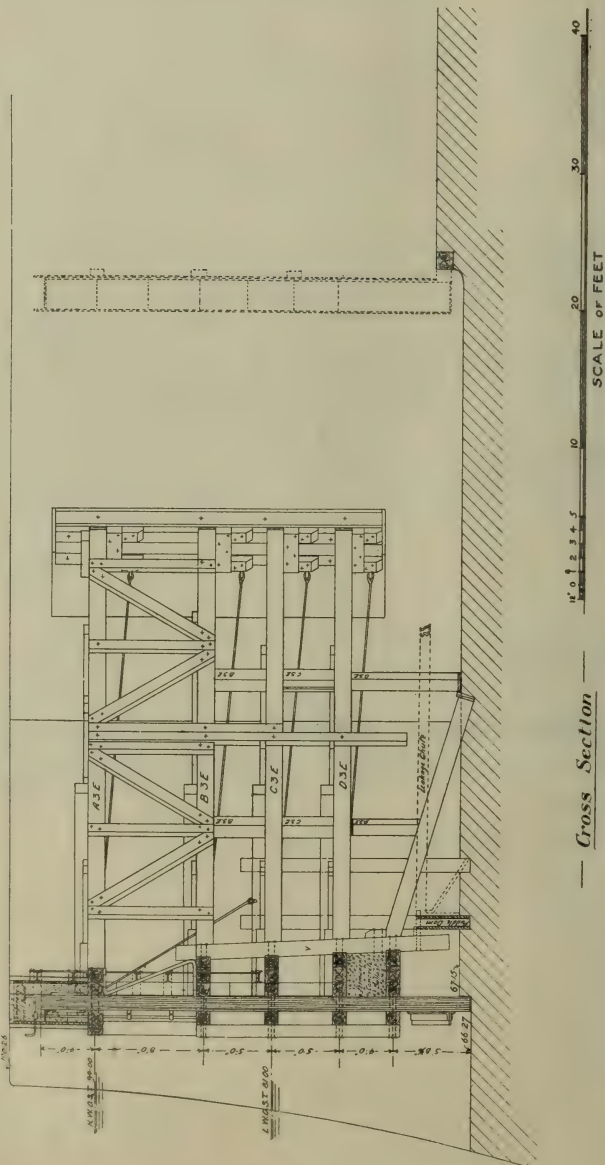


FIG. 8. L. & S.W.R. SOUTHAMPTON DOCKS: DRY DOCK No. 3: TEMPORARY DAM AT ENTRANCE AS CONSTRUCTED.

bolted to and resting against short lengths of R.S. joists which were partly buried in the brick wall by cutting out and building in. The joint between each timber and the wall was caulked with oakum to make it perfectly tight. To enable the gate to float upright, when being placed, it was ballasted with Portland cement concrete filled in between the fourth and fifth lines of walings, and to further secure it from floating up after it had been placed in position, a timber trough was built on the top of it which was filled with gravel. The gate was prevented from collapsing from the outside pressure of water, partly by the stops already described and partly by eight lines of angle struts made of Oregon pine timbers about 16in. square. The longer struts were connected together by vertical and horizontal bracing to prevent their buckling under the very heavy load which they had to support. The outer ends of these struts rested in notches cut into the gate walings, to which they were strapped and bolted, the other ends rested against vertical timber footings which were placed in large "V" shaped notches purposely cut into the dock walls, as will be described later. To secure an even bed for these timber footings cement grout was poured in between them and the wall against which they rested and to which they were Lewis bolted. To prevent possible outward movement of the gate after it had been placed, it was tied with 2½in. wire ropes and stretching screws to stout hooks let into the dock wall by drilling and grouting. The struts on the bottom waling differed somewhat from the others, being shorter and raking downwards to notches specially cut in the floor. These, of course, produced a tendency to lift the gate; on the other hand, a pair of short raking struts at each end of the gate, let into the timber stops, which have already been described, served to prevent its upward movement.

While the gate was being built the two "V" shaped notches were cut in the gate recesses, and the footing timbers described above were fastened into them.

As these notches projected many feet below low tide a limpet dam was used to exclude the water while cutting them. This is shown on drawing No. 9. It consisted essentially of a box containing three sides and bottom sufficiently stiffened and ballasted. The box was placed against the wall to be cut away in such a position that the wall formed the fourth side of the box. On pumping out water from the box the latter was of course pressed against the

wall by the weight of water outside it. As will be seen from Fig. 9 the dam was constructed of 6in. timber sheeting, the joints being caulked, and the whole stiffened with frames constructed largely of old rails, so as to provide the necessary strength and weight. The edges of the sides and bottom which met the wall were made watertight with a long padding of oakum sewed up in canvas tubing. This little limpet dam or box had an inside length of 13 feet and a width of 4½ feet and thus afforded ample room for the masons to cut away the notch in the wall. The dam was placed in position by a floating crane, and after being pumped out remained very nearly dry. A slight leakage of water was removed by a 1½in. ejector fed with steam from a small vertical boiler. To avoid any possibility of the limpet dam accidentally floating up, it was tied to strong Lewis eyes let into the wall by wire ropes and stretching screws, and was ballasted by rails standing upright on the floor. After the notch had been cut in the western wall and the footing timbers fixed and grouted, the limpet dam was removed and fixed on the eastern side, where the same work was carried out, after which it was removed entirely. It would, of course, have been impossible to open the dock gates properly while the limpet dam was in position, but fortunately a ship was in the dock at the time undergoing a long repair, so that no interruption to traffic was occasioned.

Meanwhile the big gate was built on its flat, inside upwards on an adjoining quay. It was packed up on temporary timbers about 5ft. high from the ground so as to enable the workmen to get underneath it. The timbers were mostly handled by means of a 3-ton crane travelling on a special track which was laid up to the gate. When completed in April, 1916, the gate was launched just like a ship, by tilting up the landward end of it and then allowing it to run down special timbers over the edge of the quay wall into the water. Before launching it, however, the question of how it could be made to float upright so as to easily bring it into position at the dock entrance was carefully studied. Calculations were made as to its flotation, and these were confirmed by experiments on a model of the gate made to a scale of ½in. to a foot. This model was immersed in a tank and ballasted in various ways. As a result of these experiments it was decided to introduce the concrete ballast between the fourth and fifth walings, which

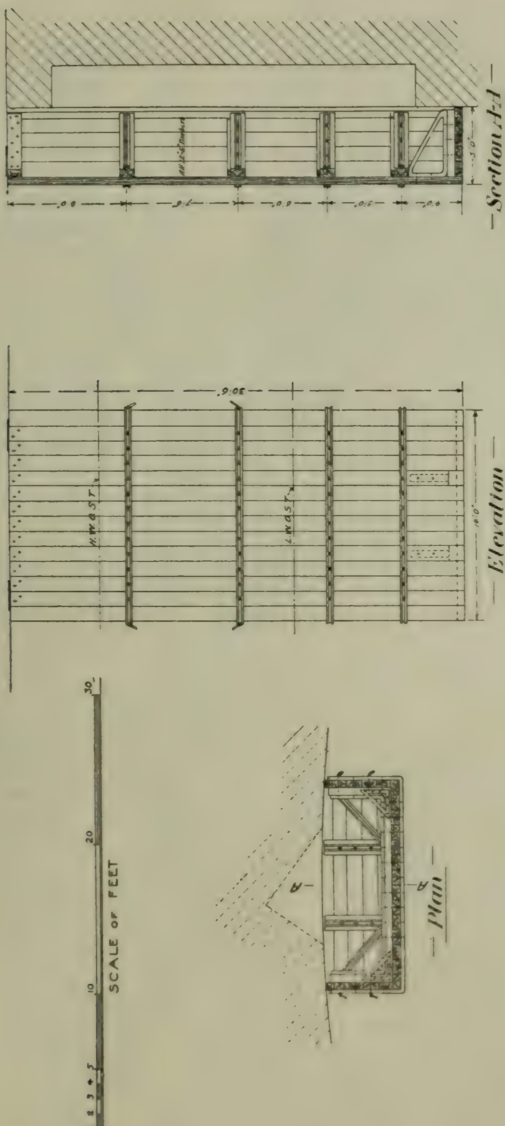


FIG. 9. L. & S.W.R. SOUTHAMPTON DOCKS: DRY DOCK NO. 3: LIMPET DAM AS CONSTRUCTED.

has already been mentioned. When the dam was first launched it floated on its back, the concrete was then poured in, the gate being meanwhile prevented from sinking by suspending its lower end from a barge. As soon as the concrete was partially set the gate was let go, and, as the model experiments predicted, it then took an almost upright position on the dock bottom. To float it into the dock entrance it was lifted by two barges, one at each end. Each barge was provided with lifting apparatus consisting of a 5-ton hand winch hauling a steel wire rope which passed through a pair of 4-sheave purchase blocks, one of which was made fast to the barge, the other being attached to a wire which passed over a cathead projecting from the bow of the barge and which was secured to the gate. Each of these tackles were capable of lifting 30 tons, which was more than sufficient in view of the fact that the gate as ballasted only weighed about 110 tons, and that most of this was borne by flotation. With these tackles the gate was lifted a few feet at high tide, and the barges were towed round to the dock entrance, and placed in such a way that they pushed the gate before them into position against the stop. The gate was then lowered down, and the barges being removed, it was made fast with the wires and stretching screws previously described, the lower ones being coupled up by divers.

The next operation was to insert the long angle struts. Until the first two lines of struts were fixed the tide had to be allowed to pass in and out of the gate through the sluices which were kept open for the purpose. To diminish the quantity of water passing as much as possible the permanent dock gates were closed and the dock emptied, so that only the space between the iron gates and the temporary gate was filled with tidal water. Before erecting the first two frames, each of the top struts was connected by the vertical bracing with the one underneath it. This saved labour and prevented the long timbers sagging by their own weight. The struts were thus built up in pairs on the quay and lowered into the water behind the dam by cranes, they were then floated into their proper position and wedged up tight and strapped securely. After the first two frames of struts had thus been fixed the sluices in the gate were closed at low tide so that water should no longer pass through it. At the same time the dock itself was flooded up to low water level and the permanent gates opened;

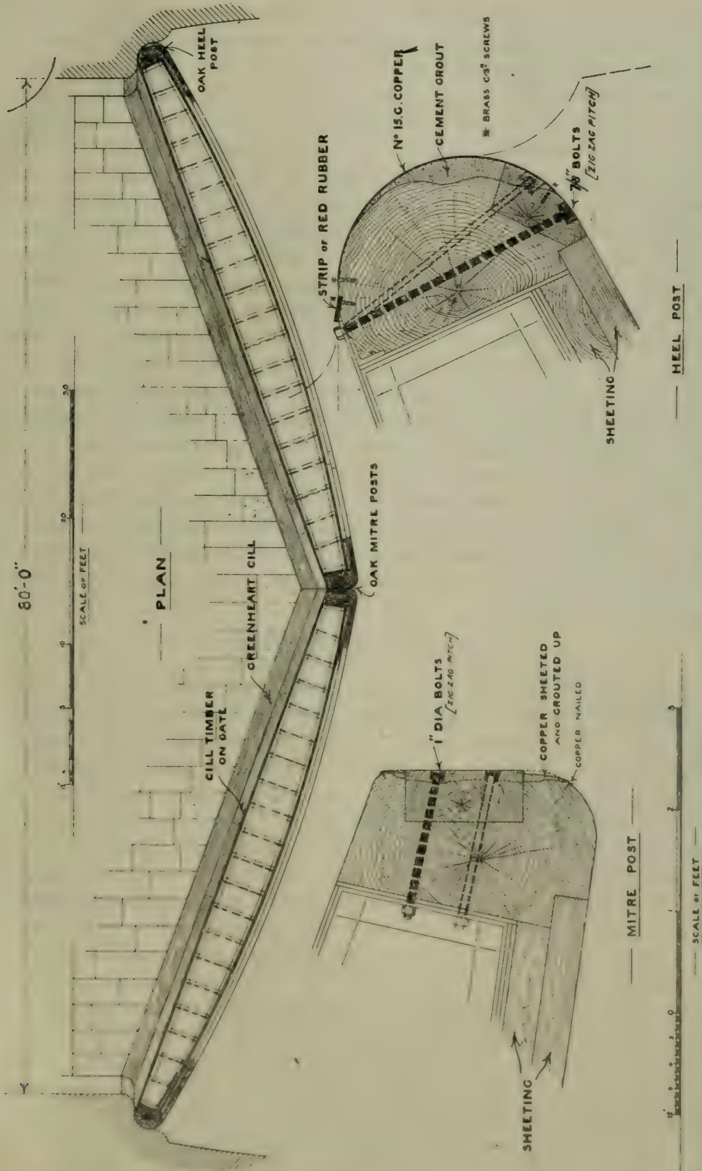


FIG. 10. REPAIRS TO GATES.

the dock was then immediately pumped down to a level just below the third frame of struts in order to allow these to be fixed. The object of flooding the dock was to maintain the water inside the gate at a practically constant level. Had the dock not been flooded the water inside the gate might have quickly leaked away through the permanent gates into the dock, and then the pressure of water outside would have burst the gate, which at that time was only partially strutted. As it was, the third frame of struts was got in without mishap, and the fourth frame in a similar manner. The dock was then pumped out dry, after which the fifth frame of short struts was put in. The latter frame was hardly needed, but served to relieve excessive pressure in the case of an extraordinarily high tide. The gate was found to be very fairly tight, and after some caulking had been done by diver the quantity of water leaking through was reduced to about 20 gallons per minute. In order that this water should not spill over the entrance floor and interrupt the work, a low puddle dam was built about five feet away from the gate, and the leakage water which accumulated between the two was carried by a small trough over the dock cill into the dock itself, where it ran away into the main pump wells.

REPAIRS TO GATES.

As mentioned above, the dock gates were probably over 60 years old. They are shown to some extent on Fig. 10. They were built for the most part of wrought iron, the decks and inner skin with the end stiffeners being constructed of this material. The outer skin was formed of two layers of 4in. fir planking. The heel and mitre posts were of oak and the cill timbers of greenheart. The weight of each gate was taken at the hinged end on a cast iron heel piece resting on a pivot of the same material, and at the front end on a cast iron roller running on a cast iron segmental path. The iron plating had got very much rusted, especially on the upper surface of the decks where dirty water due to condensation and leakage was continually lying. The under sides of these decks were in quite good condition, but the iron skin was rusted away in places. A certain amount of patching was done with a view of strengthening it. The outer thickness of the timber skin was eaten away to some extent by marine insects, so it was covered with a third thickness of $1\frac{1}{4}$in. fir planking.

The cast iron rollers had become very much worn, allowing the gates to droop forward; these were renewed entirely. The greenheart cill timber was in good condition, and nothing was done to it beyond re-facing it. The worst deterioration was in the oak heel and mitre posts. The latter were badly eaten by marine insects below low water, so much so, that they no longer touched when the gate was closed, but showed a gap of over 1in. They were repaired by attaching two new oak face pieces about 14in. wide and 6in. thick. These were housed in to the old posts and bolted through to the ironwork of the gates. One of the heel posts was also badly eaten. This was repaired partly in the same way, viz., by cutting out the damaged parts and bolting on a new timber. At the back of the post, however, this could not be done as the post rested in the hollow quoins and was not accessible. It was therefore repaired by covering it with copper sheeting 1/16in. thick, passed right round the post and screwed to it. Cement grout was afterwards poured in between the copper and the wood to fill the voids eaten away by the insects. It is interesting to note that while both the fir and the oak were badly attacked by these insects, the greenheart was quite unaffected. This is in accord with the experience of other structures in sea water at Southampton.

It was, of course, realised that these repairs being of a patchwork nature could not ensure an absolutely watertight fit between the heel posts of the gates and the hollow quoins against which they pressed. Accordingly, to minimise leakage, a strip of indiarubber 3in. × $\frac{3}{8}$in. was let into and screwed on to both heel posts from high water level down to the bottom of the gate. The effect of this has been excellent and the leakage through the gates, which formerly amounted to over 500 gallons per minute, has been diminished to about 150 gallons per minute.

REPAIRS TO CILL.

As previously mentioned, the gate cill is constructed of Bramley Fall sandstone, the stones being dressed true and laid as voussoirs of an inverted arch. Each stone is about 2ft. 2in. wide on the top surface and probably about six feet deep. The gates do not press directly against these stones, but against a greenheart timber about 15in. square, which is let into a rebate of the same dimensions cut into the sand-

stone. The arrangement is shown on Fig. 11. The timber was fastened down by screwed bolts secured to Lewis shaped nuts, which were cemented into holes cut in the sandstone. It was not fastened horizontally in any way. In August, 1915, after the docking of a ship, it was noticed that one length of this timber had risen about an inch and that it was split, and it was rightly guessed that the holding down bolts had in some way given out. Eventually it was found that the Lewis nuts had burst their way out of the stone and that this had occurred practically throughout the whole length of the cill. In fact had it not been for the inverted arch shape of the cill this timber would no doubt have risen

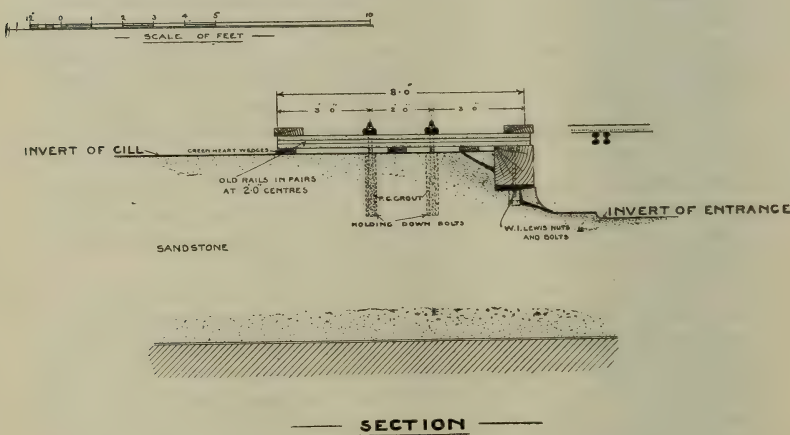


FIG. 11. TEMPORARY REPAIRS TO CILL.

up long before. It was noticed, too, that the surface of the sandstone at the back of the timber which had risen was fractured in the manner shown in Fig. 11. Subsequent investigations showed that this damage was primarily due to the bad state of the gates. The fact that the mitre posts were eaten away, and did not meet properly, prevented the gates from supporting each other, and threw a greatly increased pressure on the cill. Moreover, the rollers being worn away allowed the gates to droop while they were being shut, but directly the dock was pumped out the water pressure caused them to lift again, and by friction to drag

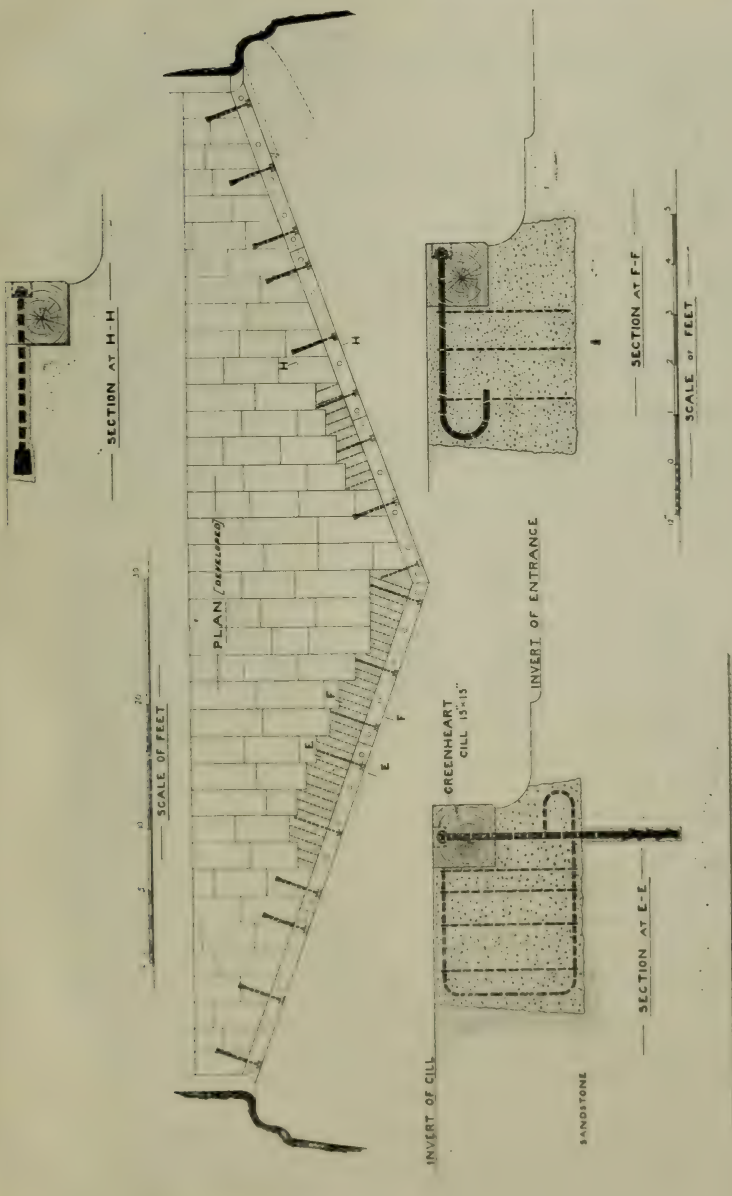


FIG. 12. PERMANENT REPAIRS TO CILL.

the cill timber up with them. Thus the timber was subjected to an upward pull and a horizontal pressure which finally burst out the holding down bolts and crushed the stone behind it.

At the time the accident occurred it was very essential to keep the dock in use for a while. Arrangements were made, therefore, to prevent the displaced cill timber from lifting further by anchoring it down in the way shown in Fig. 11. Short lengths of old rail were laid in pairs across the timber at intervals of about two feet, and these were anchored down by Lewis bolts let into the undamaged sandstone and grouted up with neat cement. This arrangement served to keep things safe until the dam was built and the dock entrance dried out, when more satisfactory repairs were undertaken. These are shown in Fig. 12. It was at first intended to cut out the sandstone of the damaged cill and substitute granite. This, however, would have been a difficult and perhaps a risky operation, as in doing so the underlying lime concrete which was suspected to be by no means watertight, would have had to be exposed. Accordingly it was decided to leave the sandstone as it was, but to take out and refix the greenheart timber, renewing any lengths which were imperfect. The new timber was firmly anchored down by means of Lewis bolts projecting at least two feet into the sandstone. It was considered that this would give a far better anchorage than the short Lewis nuts which were previously used. These vertical bolts were placed at intervals of about 2ft. 3in. Moreover, a few horizontal Lewis ties were introduced to prevent any possible rocking of the timber. The space between the timber and the stone was then carefully grouted with neat cement. Before the timber was fixed the sandstone voussoirs, which had been cracked, were cut away a depth of three feet and for about the same length. The voids thus formed were filled by Portland cement gravel concrete $1 : 1\frac{1}{2} : 3$, strengthened with steel bars as shown in Fig. 12. It will be seen that this concrete was anchored down at intervals to the undisturbed sandstone beneath it by Lewis ties let into holes in the sandstone and grouted with neat cement. The cill as repaired has proved to be quite watertight and satisfactory.

It may be mentioned that the question of adding sand to the cement grout was considered in order to minimise the possibility of its shrinking. An experiment was made,

however, which showed conclusively that sand grout is not to be relied upon. A quantity of sand cement grout was poured into a vertical hole bored between two timbers in such a way that they could be separated and the grout examined after the latter had set. It was found that the cement had come to the top where it was practically free from sand; half way up it was a mixture; at the bottom it was pure sand and quite useless.

The works were completed in September, 1916, when the dam was removed in just the same way that it had been built, but of course with far less difficulty. After the struts had been taken out frame by frame, the gate was lifted off by one of the barges which had placed it and towed away and beached.

The works were designed and carried out by the Docks Engineer's Department at Southampton. It may be of interest to give the cost of the various items, but it must of course be remembered that war conditions have greatly increased both wages and price of materials.

	£
Flattening floor and building compensation walls, etc.	3,980
Building and erecting and removing temporary dam, including the limpet dam	2,371
Repairs to gates and cill, and other minor works	1,327

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The Rational Design of Reinforced Concrete Wharves and Jetties, with particular reference to those for Wet Docks that have a permanent Water Level.

By W. CLEAVER, M.Inst.C.E.

(This publication is slightly abridged from the original paper.)

The title of my paper may at first sight appear to be rather ambitious, or even dogmatic, as if there were very few wharves, etc., designed rationally, but nothing is farther from my mind. I use the word rational advisedly, and in its extreme literal sense; in other words, the design of reinforced concrete wharves from a common sense, or practical standpoint, based on my own personal experience of about 12 years on this particular class of work.

In my opinion the great fault usually found with designers of wharves, jetties, etc., is that they do not take a broad enough view of the matter at the commencement, and thereby omit to take into account the combination of circumstances which, to a more or less extent, affect the design. For instance:—

(1) Vertical loads on the wharf, *i.e.*, point loads or distributed loads, as the case may be.

(2) Amount of impact stresses from ships when berthed against the face.

(3) Pressure of filling at the back of the wharf, or thrust of natural ground according to circumstances.

Nature of strata for pile driving, etc., and particularly

(4) The question of moorings; and

(5) The general effect of the dredging on the design of the structure.

One or other of these items is often unconsidered, and the item mostly overlooked is the question of

(6) The method of dredging and its relationship to the general design of the work.

If some of the points mentioned, or similar ones, are overlooked in the case of well-established and understood methods of construction, such as timber wharves, etc., how much more easily are they lost sight of when the work is constructed of reinforced concrete, and when the designer (although perhaps a specialist in reinforced concrete) may have very little knowledge of the many and varied conditions to be met with in the actual construction and use of such wharves, etc., such knowledge being absolutely essential, especially in work of this class.

Endeavour will be made, therefore, to treat the subject under various headings, in almost the same rotation as would occur in actually carrying out the work.

DREDGING.

One can almost take it as a foregone conclusion that in most docks where jetties or wharves are required the first thing to be faced is to dredge out the berth. The site may first of all be dry land, to be dredged in its entirety, or it may be inundated to a depth of 5, 10, or more feet, but the operation would be almost the same in either case if the subsequent depth was to be such as required for a modern berth of, say, anything from 30 to 40 feet depth of water.

Except in but very few instances, the work must be carried out with a bucket dredger, of the hopper type or barge-loading variety, and the engineer's first trouble is due to the method of disposition and manipulation of the dredger with reference to the section desired.

The dredging contractors, if the work is done by contract (and often the dredging master, when the work is done departmentally), will invariably attempt to carry out the work with the dredger placed parallel with, *i.e.*, broadside to the proposed face of wharf, and admittedly this is the easiest way as regards moorings, etc., and particularly

as regards the placing of barges when a barge-loading dredger is used.

It is, however, the very worst position with reference to the carrying out of the dredging to the best subsequent advantage, and in accordance with what the final section should be if properly designed.

Diagram No. 1 indicates the section of the dredged cut which should be aimed at, and those responsible for the

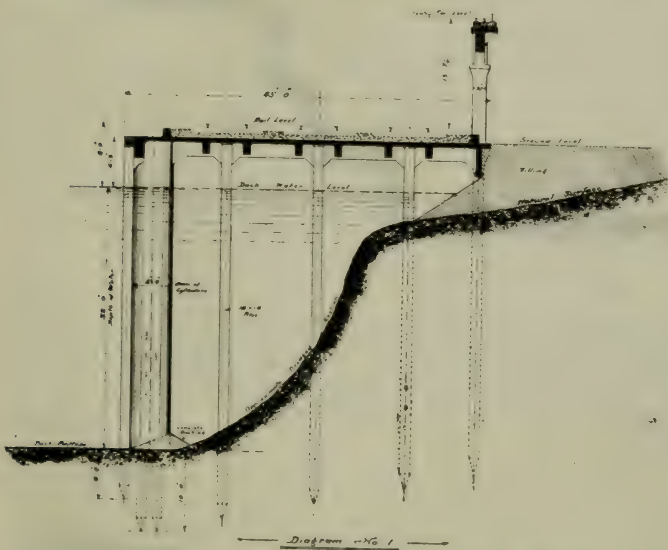

Diagram 1. Mouchel-Hennebique System.

dredging will generally demonstrate in the most plausible manner how easily this can be carried out in a series of cuts something on the lines shown on Diagram No. 2.

This is, however, the time for the engineer to object. I am quite prepared to challenge any dredging contractor to guarantee the result, as I have known to my cost how impracticable the method is, and what little reliance can be placed on the resulting section being anything like the one anticipated. The slightest carelessness or oversight on the dredging master's part results in a section similar to that shown in dotted lines on Diagram No. 2. I therefore now insist that when dredging on or near the actual line of wharf

the dredger must be placed at, or nearly at, right angles to the face-line so that the buckets will cut the strata something in the manner shown on Diagram No. 3.

In this way it can easily be seen that even if the depth

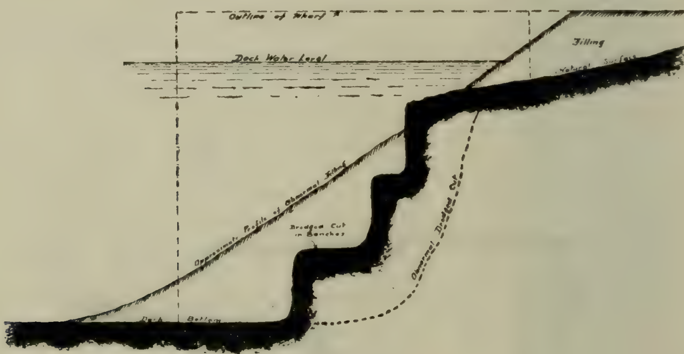

DIAGRAM 2.

of dredging necessitates two or three cuts, it is much more feasible to so manipulate the dredger that the buckets will keep truer to the line of sights, and the natural cut will more nearly approach the profile desired.

DIAGRAM 3.

If the dredger sways to right or left, as the case may be, when placed broadside, the buckets cut out of line immediately, with possibly serious results, but when placed

at right angles any similar swaying causes simply a radiating cut, which is always well within the line of cut fixed. It is almost needless to mention in this connection, and as proof of the argument, that any dredger is far more liable, however efficient the head, stern, and side moorings may be, to sway to port or starboard than fore and aft.

If the dredging is carried out in benches as the result of the broadside method, it also causes the contractor for the wharf a great deal of worry and expense, owing to the difficulty of pitching the piles if the centre of same happened to coincide with the edge of the cut, which causes the pile point to kick out, and as a result it is almost an impossibility to pitch the pile correctly.

With a cut similar to that shown on Diagram No. 3, which can easily be carried out, or, at any rate, very nearly approached by the end-on method, no such trouble is usually experienced in pitching the piles.

It is well worth the extra trouble, if any, only to avoid the inevitable bad language accompanying the operations due to the repeated insistence by the engineer to pick up and re-pitch the piles.

When the dredging is done properly the filling, as shown in Diagram No. 1, is normal in quantity and safe, whereas after an oversight, as shown by the dotted cut in Diagram No. 2, the filling would be, as shown, hatched, and when any reader has, like myself, had the experience of the trouble of supporting this abnormal and unnecessary filling with its kindred and consequential troubles, such as grabbing the surplus toe, etc., he will bless (?) all dredging contractors with their perpetual promises, which materialise only on paper.

The moral is either insist in your specification that the work shall be done by the end-on method, or carry out the dredging work departmentally with your own plant and staff.

PILES.

The next item we have to consider is the question of piles, and in the first place I unhesitatingly advise that for modern requirements concrete piles, having a section less than 14ins. square or equivalent area, should never be used if over, say, 30ft. long.

It can no doubt be easily proved theoretically that

12in. square piles in many cases are quite large enough, but when it comes to actual practice it will be found that the extra cost of 14in. piles is more than compensated for by the extra convenience in handling, *i.e.,* the lesser risk of bending and similar damage, coupled with the fact that in the designing of the work advantage can be taken of the additional sectional area of the piles to increase the span of the bays and the individual loading of the piles.

The point I wish to impress is that 14in. square is a far more satisfactory standard section to adopt for reinforced concrete piles than 12in. square, no matter what the work is like or intended for, as the general design of the structure should be based to a great extent on the section of pile decided on in the first place.

As to whether the piles are square, round, or octagon, etc., is quite immaterial, but only that the gross sectional area shall be about 196 sq. ins. in preference to 144 sq. ins.

Regarding the design of reinforcement for piles, the author has no particular predilection on the matter, as he does not think there is very much to choose between any of the designs now on the market, as they have their own particular advantages and disadvantages; for instance, if I had any preference at all, I would, from a theoretical standpoint, vote in favour of round piles, with a fair number of bars of comparatively small section, in preference to square piles with, say, four bars of large section. On the other hand, square piles are so much easier to make and inspect that, considering how slight is the disadvantage in theory, I would prefer the square ones in practice.

I will say this, however, that after considerable experience and severe tests, I consider the ratio of sectional area of steel to total sectional area of piles should not be more than about 36 to 1, which would give six super inches of steel in a 14in. square pile. This is, of course, for vertical reinforcement only, exclusive of any wire ties, binding, etc.

PILE SHOES.

This is another very debatable point with some people —for instance, as to whether the shoe should be of uniform taper or varied, also whether sharp-pointed or blunt-nosed, whether the vertical reinforcement should rest on the shoe or not, etc., etc.—but I have always adopted one design

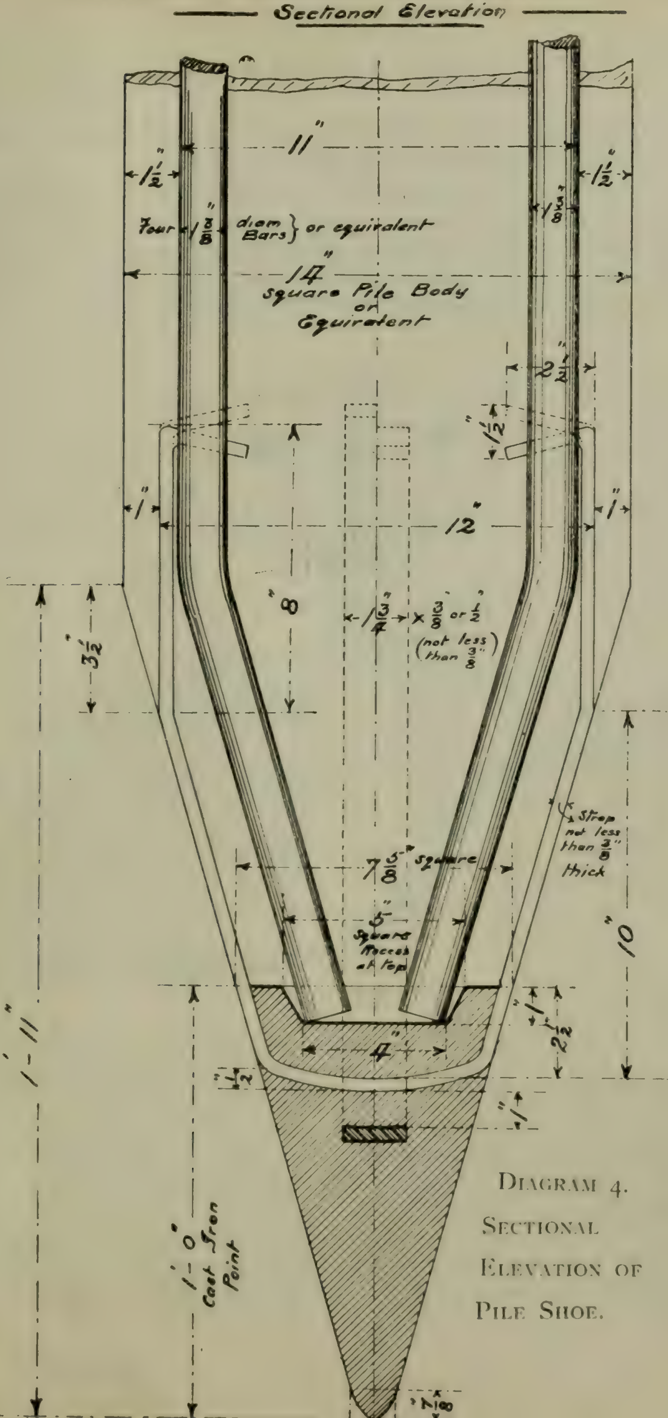

DIAGRAM 4.
SECTIONAL
ELEVATION OF
PILE SHOE.

of shoe for all circumstances, with very satisfactory results, as per the following particulars :—

"The shoe to be of uniform taper, the faces being at an angle of about 16 degrees to the centre-line of the pile, with nose slightly blunted, as shown on Diagram No. 4. The length of the cast-iron portion to be about half the total length of the taper, the area at top of

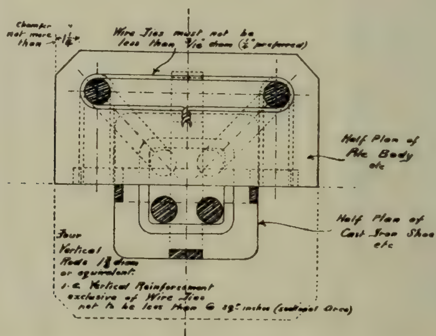

Diagram 4a. Plan of Pile Shoe and Pile.

the cast-iron point being about 40 per cent. of the sectional area of the pile in the case of the 14in. square piles. The vertical rods to rest on the cast-iron point and to be kept in place by the recess, as shown. The straight portions of the wrought-iron arms to always be embedded in the concrete of the pile and not kept flush or project from the body."

HELMET.

The author has always held the opinion, and still does, that a helmet should in every instance be used when driving reinforced concrete piles, irrespective of whether the piles can stand the driving without same or not. A concrete pile should never be needlessly punished in the preliminary driving, as it is time enough for hard driving when the pile is almost home.

By that time the cushioning material in the helmet, which has done its duty in saving the pile until the right moment, is hard enough for all practical purposes, and there

can be no appreciable difference in the driving, with or without the helmet, by the time the final sets are reached.

When the pile is pitched and constitutes an excessively long column with reference to its section the helmet serves the purpose of taking up the bulk of the abnormal vibration, etc., due to this, and which would otherwise tend to damage the pile before it had been driven far enough to attain the requisite rigidity.

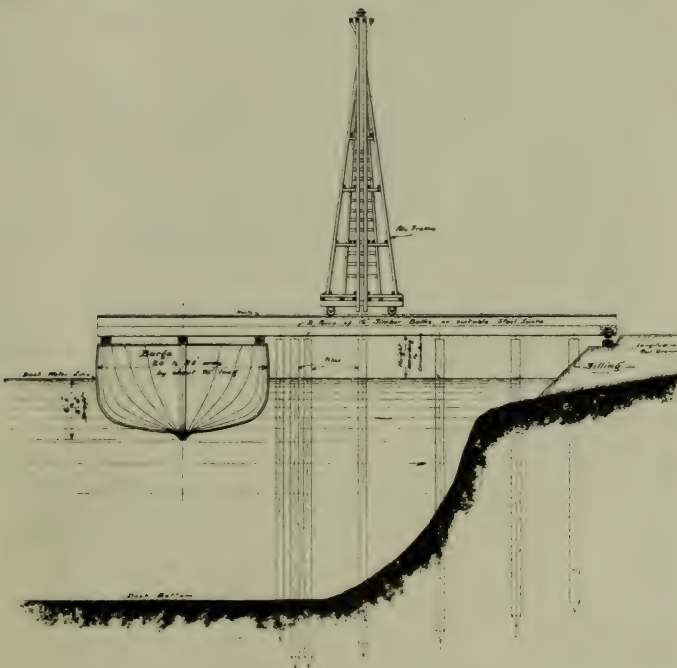

DIAGRAM 5.

METHOD OF DRIVING PLANT, ETC.

Except in the open sea, etc., where any other method is almost impracticable, I have always strictly objected to driving piles off a floating barge which is secured only by means of rope moorings. On the other hand, it is manifest that a timber-piled temporary staging adds very greatly to the cost of the work, both in material and time.

The practice of driving the back row of piles first of

all off the shore and then using same as a support for a stage to drive the next row, and so on, has many objections, not the least of which is the probability of damaging the piles, due to the inevitably long cantilever of the temporary stage or framing.

To overcome these various objections, the arrangement shown on Diagram No. 5 was designed by the author. It is a sort of combination of floating barge and fixed stage without the disadvantages of either scheme. This method has proved not only very satisfactory in service, but is the cheapest of the lot. It permits of the use of a floating barge without its mooring objections, and consequent instability. It also avoids the use of any of the permanent piles in a temporary capacity. In addition, it enables the pile-driving to be completed bay by bay, and leaves the work clear for the superstructure to be proceeded with at once without having to wait for the removal of any temporary work, and, what is most important of all, the piles can be driven to a better and truer line than by almost any other method.

As regards the class of pile-driver to be used, I have always held the opinion that an ordinary drop monkey, weighing at least two tons, with a short drop of about five feet, manipulated by means of the usual hand nippers, is the most efficient.

The only reasonable objection to this is the fact that the operation is a slow one, owing to the human element, *i.e.*, the difficulty of obtaining a combination of plant and men of the class that will produce the requisite number of blows per minute. Hence my repeated statement that the endless-chain design of pile-driver is by far the best on the market, as it does away with the tediousness of the hand nippers, and yet provides a monkey with a clear drop.

Steam pile-drivers, friction winches, etc., will not bear comparison for one moment, for the reason that so much extra allowance has to be made, in the way of additional driving, etc., in order to compensate for the loss in friction, and this, of course, must take up extra time, and one can never be so sure of the actual net result of the driving.

Whatever pet formula may be used for calculating the supporting power of any pile driven by the ordinary or by the endless chain machine, one can always rely on the effective or dead weight of the monkey being calculated upon, but what the net effective weight of the monkey is in

the case of steam monkeys or monkeys attached to wire bonds and friction winches, is often a matter of pure conjecture.

DESIGN OF WHARF.

The present paper deals primarily with wharves for wet docks with a permanent water level, and for which bracings under water would present almost insuperable difficulties. One has, therefore, to depend almost entirely on the unsupported piles below water-line, and after trying various designs, the author has come to the conclusion that single-pile structures, that is, designs consisting of single independent pile supports throughout, are not very satisfactory.

It must be realised that modern ships in midship section are almost square in profile, and when berthed are liable to come in contact with any portion of the wharf-face, from water-line to almost dock bottom.

If, therefore, it was possible to introduce effective bracing a short distance below the water-line, what would be the use of same, as it would only be of service in resisting any swaying tendency in the wharf, which tendency should be almost non-existent if the structure is properly anchored by means of back piles or counterfort wall.

The net sectional area of the piles below the bracing to resist shear due to impact of the vessels (*i.e.*, the most important stress to be provided for) would still be the same.

Of course, piles of extra large section could be used, and would no doubt be effective, but the inertia of such piles, when made of concrete, increases to such an extent as compared with piles of normal section that the difficulties of driving same are by no means easily overcome.

For these reasons the author has reverted to the old original method of driving piles of normal section singly or in groups of two or more, according to circumstances, loading, etc., and then encase these with reinforced concrete cylinders and mass concrete hearting. The resulting column imposes such a large sectional area throughout its depth against shear that no fear need be entertained as to its capabilities to resist the impact even of the largest vessels.

These columns, coupled with suitably designed beams, also enable the spans at the front of the wharf to be made longer than would be the case if single piles were used.

The extra dead weight also adds materially to the

rigidity of the wharf, a very important acquisition when the depth is 30ft. or over, and in addition the designer is able to arrange the fixing of the various mooring-rings, bollards, etc., in a much more efficient manner by attaching them at or near the cylinder columns.

Cylinder designs have no doubt got into considerable disrepute, particularly in contractors' minds, owing to the difficulty of making the rings watertight and pumping out the interior. If the cylinders are sunk in water-bearing strata, then there should be no great difficulty, but if the bottom consists of gravel, it is practically impossible to get the interior dry, and in my opinion it is folly to attempt it. The concrete hearting can be placed in the water with quite satisfactory results if properly mixed and deposited through steel tubes with bottom doors, and a little common sense exercised in the use of same.

Cylinder designs have the additional advantage that the superstructure can be designed as a single-deck scheme without any bracing or a subsidiary series of beams near water-level.

Diagrams Nos. 1, 7, 8, 9, and 10 show some of the various sections adopted from time to time at Port Talbot, which clearly indicate the differences explained above.

The subsidiary beams and bracing add very considerably to the cost of the work, and in my opinion more than balance the extra cost due to the making and erection of cylinders in the alternate scheme. It is also very difficult to construct the joints between the beams and piles, etc., when they are situated so near the water-line, and unless very great care is exercised, this may later on be a source of considerable danger, as it is in just such places that defective joints are likely to occur, and with variation of water-level or wash from any cause, the water would soon find its way into the cracks, with the inevitable result of serious deterioration of the steel.

All this kind of trouble is avoided in the single-deck designs.

SURFACE OF DECKING, PERMANENT WAY, Etc.

For heavy structures subject to overturning moments, etc., such as cranes or gantries, and which have to be clipped to their permanent supports to prevent this, by far the best type of permanent way is, of course, the

longitudinal timber method, with the timbers well secured into the beams of the wharf and the flange rails clipped (not spiked) to the timbers.

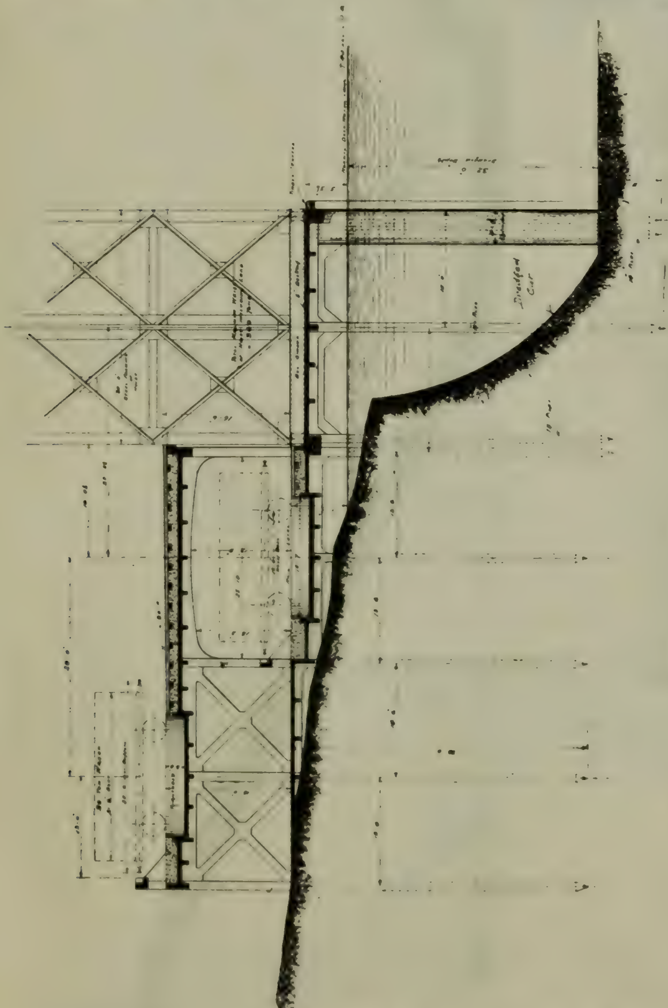

DIAGRAM 7. MOUCHEL-HENNEBIQUE SYSTEM.

For ordinary locomotive and waggon roads, however, I much prefer to use the ordinary cross-sleepered type of permanent way, ballasted in the usual manner, and I there-

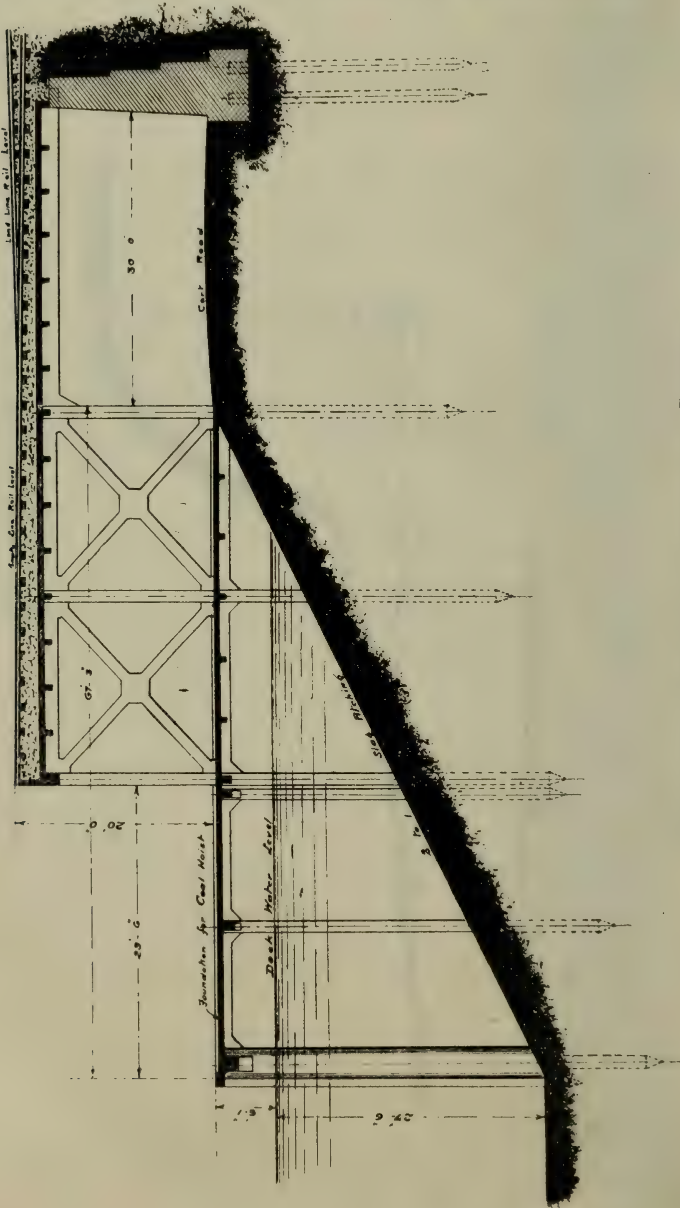

DIAGRAM 8. MOUCHEL-HENNEBIQUE SYSTEM.

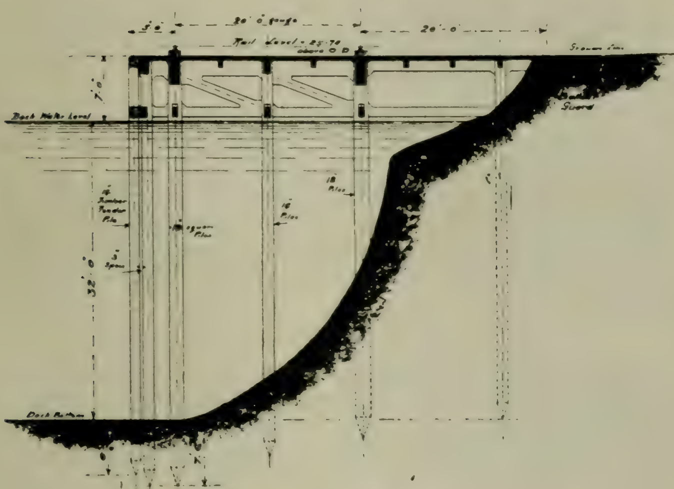

DIAGRAM 9. COOPER-HOLLOWAY SYSTEM.

fore design the decking so that the surface of the wharf constitutes really the equivalent of the ordinary formation level.

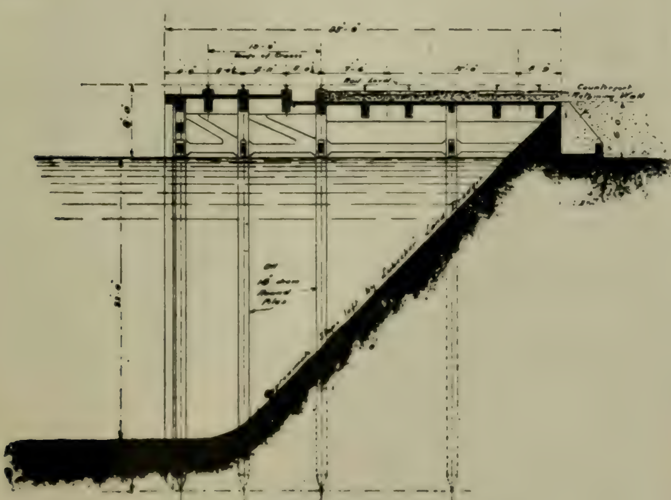

DIAGRAM 10. COIGNET SYSTEM.

To make this method a complete success, ample provision must be made in the decking for drainage by means of holes about 2½in. to 3in. diameter at suitable places, say in the centre of each bay.

If the exact positions of the rails are known beforehand, all well and good. The beams can then be arranged under the actual rails in those positions, but the arrangement just mentioned is such that the position of the sidings can, within reasonable limits, be varied laterally (without in the least affecting the stability of the wharf) by taking advantage of the spreading or cushioning effect, due to the ballast and sleepers.

FENDERS.

I have always adopted and believed in the utility and economy of protecting all working faces of concrete wharves and such-like structures with timber fendering, owing to the inevitability of same being damaged by abrasion, etc., and requiring renewal, and it is far easier to renew timber under such circumstances than concrete. For the same reason I always adopt timber piles, and also longitudinal caps, on the principle first of all (as regards using piles) that no timber fendering should ever be secured to the concrete below water-line, all fastenings being built into the superstructure only. This has the advantage of preventing any tearing or fracture of the concrete below water should the fendering happen to be torn bodily away (as is sometimes the case) by collision, etc., and it also avoids during construction any costly diving work in bolting up and similar items. The advantage in using longitudinal timber caps is as follows:—They first of all provide a uniform sliding surface for ships when berthing alongside, whereas without caps portions of the ships, plate-edges, etc., are liable to come into contact with the vertical arrises of the piles, with the consequent splitting and stripping; and secondly, they provide an additional width of a foot or so in whatever gangway is provided between cranes and face of wharf, coupled with a true uninterrupted face-line in lieu of the dangerous gaps between the piles when vertical timbers only are used.

BACK OF WHARVES.

If the strata on the site consists of good ground and the surface is above or near water-level, then I prefer to

design the back of the wharf as a counterfort retaining wall. In the first place, it is a cheaper mode of construction than piles, and owing to the extra superficial area offers more resistance to impact and lateral movement of the wharf than when same is wholly supported on piles. The condition of the site must, however, of necessity govern the choice between wall and piles, but I simply point out here as a fact based on the result of actual experience with both methods that when the site permits of same the retaining wall is the best design. Diagram No. 10 indicates the class of retaining wall referred to.

WORK ADJACENT TO WHARF.

The design for a wharf of the character we are discussing should always include any supports for adjacent structures, such, for instance, as the piers for supporting gantry, as shown on Diagram No. 1, as this adds not only to the general stability of the wharf, but is also cheaper and more satisfactory than building a separate brick wall or piers at the back of the wharf, and, in addition, the combination ensures a more correct and permanent relative distance between the line of gantry and face of wharf, etc.

WHARF DETAILS, Etc.

Without desiring to enter into any controversy regarding the claims of various specialists, or as to the wisdom or otherwise of employing specialists, etc., the author has been requested to give his opinion on the matter.

First and foremost, I believe in employing specialists if the work is of a reasonably extensive character on precisely the same principle as a general practitioner in medicine calls in a Harley Street specialist or a solicitor when he briefs a K.C., and many other similar precedents could be cited. It stands to reason that, however competent an engineer may be as regards general experience, he cannot as an individual hope to cope with a staff of experts such as the better class of specialists employ in the working out of the general details of the reinforced concrete portions pure and simple. An engineer in usual practice has to have a very broad general knowledge of numerous subjects, and it is unreasonable to expect that he should be as conversant with any one particular subject (especially such a compara-

tively new one as reinforced concrete) as those who do nothing else all the year round but to specialise in that one branch.

As to the general design of the wharves, etc., unquestionably the whole thing should be completed by the engineer responsible for same, and the reinforced concrete specialists need not be consulted on the matter at all, but after the general design has been completed and the various loads, etc., decided on, who should be the most competent to decide on the net amount of reinforcement necessary, the best sizes and proportion of beams for those loads, etc., than specialists who are in the thick of that sort of work every day?

On the other hand, I will say this. An engineer or an architect, as the case may be, should always have a sufficient knowledge of the principles of reinforced concrete calculations to be competent to make preliminary calculations and provisionally design the sizes as well as the distribution of the beams, etc., in his general design before same is submitted to the specialist. The latter can then bring his mature experience into play, and advise where and how, if any, the sizes, etc., of the various members could with advantage be modified so that the ultimate design will be absolutely up-to-date in every respect.

The specialist's staff would then also be responsible for the production of the full working detail plans of the reinforcement, bills of quantities, etc., throughout the job.

For an engineer or architect, however, to simply make a rough design and trust blindly to the specialists to do all the calculations and use their own discretion entirely as to the sizes and distribution of the beams, etc., is, of course, manifestly wrong and foolish, as the specialist cannot possibly be in the position (except in rare instances) of knowing all the local conditions which naturally must govern the design not only as a whole, but also as to most of the actual details.

Each specialist no doubt has his own pet theory as to how the various reinforcement should be designed and arranged. One specialist may put three round bars where another would put six of smaller section, or still another may prefer five square bars, so that the engineer should be in the position of having sufficient knowledge and experience to judge which of the various suggestions would suit best in his particular case, and more particularly be able to

calculate if any or all of the specialists quoting had provided sufficient total sectional area of steel to safely withstand the load specified, and at the same time be strictly in accordance with the clauses mentioned in the specification regarding factor of safety and similar restrictions.

To a great extent there is not very much to choose between the various designs of the few trustworthy systems now in vogue, it, more often than not, being a case of which of the specialists exercises the best taste in the application of his system to the particular work in question.

One point I would like to emphasise, and that is, that whatever system is adopted, I do certainly believe in the principle that all reinforcement should be as self-contained or self-supporting as possible before any concreting is done, and all important intersections wire-bound, etc., so as to avoid any risk of the various rods, shear members, or other wiring being deranged or displaced during the operation of placing the concrete, punning, etc. ; in fact, I now insist on all loose members of beams, columns, etc., being individually bound with small-gauge wire to the main rods.

In order to ensure satisfactory work, and so that reinforced concrete shall hold its own with any other method of construction, all concerned must be educated to appreciate and realise the fact that all parts and material must be made as fool-proof as practicable.

PRELIMINARY PLANT AND ORGANISATION.

The great mistake usually made is to starve the job in hand at the commencement, coupled with the failure to appreciate the fact that although it is admitted that the bulk of the work can be carried out by unskilled labour, the necessary small amount of skilled labour and supervision is absolutely vital, and must be efficient if the ultimate result is to be satisfactory to all concerned.

For instance, jigs and templates for bending rods and wires must be well thought out and well made.

Mechanical concrete mixers should be always used in preference to hand mixing. All these improvements are to the contractor's advantage in the long run, in the saving of labour, etc., and such provisions are particularly important now that female labour has been requisitioned, and, in fact, is actually engaged on the making even of concrete piles.

DAMAGES AND ACCIDENTS.

An objection often raised to reinforced concrete is the difficulty of repairing the structure after damage by collision, etc., but so far as my experience extends, this idea is a fallacious one. In the first place, the damage from collision and kindred causes is generally of a much less serious character than would be the case with timber and similar construction, and the repairs are usually easier to carry out in concrete than would be the case when stripping and renewal of timber or steel work had to be carried out.

Reinforced concrete work, such as in a wharf, is so monolithic in character (in actual fact, not in theory) that any blow due to a collision is immediately transmitted over such a wide area that a good deal of the effect of it is dissipated, and the actual damage or disruption is, as a result of this, extremely localised.

The very high inertia of a reinforced concrete wharf as compared, say, with a timber one of like dimensions contributes very greatly to this result, owing to the fact (which, by the way, is generally overlooked) that the ratio between a reinforced concrete wharf, and the cause of the collision (say a ship) weight for weight, is very much more in favour of the concrete wharf than would be the case if the construction were in timber or steel.

As regards the repairs after a collision, in most of the instances I have had experience of the ship or other delinquent came off second best, and very much so, and no repairs were found necessary in the case of the concrete, as, for instance, take the truck accident shown on Photograph No. 11.

Another instance I recollect is that of three sailing ships breaking away from their moorings simultaneously and careering down the dock, being brought up short eventually by coming in contact with the corner of one of the tip jetties. The plates of the ship nearest to the jetty, which actually came in contact with the concrete, were badly dented, but no damage of any kind could be traced in the reinforced concrete portion, whereas some damage did occur to the steel structure of the hoist which was struck by portions of the rigging, bulwarks, etc.

Another instance occurred recently to one of our latest wharves. A large steamer of about 4,000 tons was approaching the wharf, stem on, at a fairly good speed when

something went wrong with the steering gear, and before the engines could be reversed or the way reduced, the steamer crashed into the wharf approach, which happened to be the weakest portion, of course, as usual, and to still further test the work, the stem hit the front beam or curb,

FIG. 11. ACCIDENT TO WHARF AT PORT TALBOT.

almost in the centre of the bay, between the piles, resulting naturally in imposing the greatest bending moment on the beam that was possible under the circumstances.

The extent of the collision can be appreciated when I state that the stem of the vessel buried itself in the concrete of the wharf, bursting first of all through the beam itself,

which was 10ins. wide by 22ins. deep, and coming to rest 5 to 6ins. in the decking beyond the beam, as shown on Diagram No. 6. Diagram also shows very clearly the actual damage to the face of the beam.

I can, however, assure you that, incredible as it may sound, there was not the slightest trace of damage or disturbance to any portion of the wharf, other than at the actual spot, in which the ship's stem buried itself. In fact, there was not even a crack to be traced in the decking beyond a radius of about 3ft.

About the most notable instance, however, that we have had at Port Talbot occurred as recently as last January.

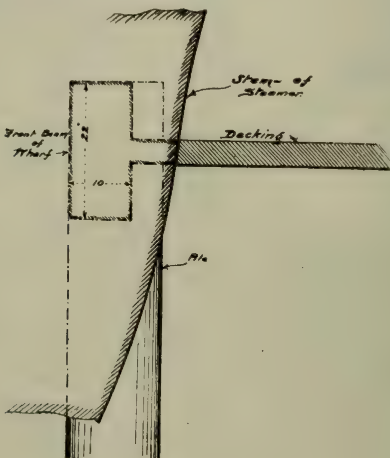

DIAGRAM 6.

By some means the stern of a steamer was allowed to collide with the corner of one of the reinforced concrete wharves, and in slewing round to get out of the predicament, one of the blades of the propeller got jammed between the timber fender pile and the reinforced concrete pile, and as the main engines were in use at the time something had to give way.

The steamer was a 4,000-tonner, with engines indicating about 1,500 horse-power, so that the strain induced by stalling engines of that power can be imagined. The main concrete pile stood, but the timber fender pile and fender

cap were split beyond repair, and a couple of the concrete beams and braces were slightly damaged, *i.e.*, cracked.

The repairs and renewals to the timber work cost about £42, and the repairs to the reinforced concrete only about £8, making £50 all told.

If it had been a timber jetty in all probability a considerable portion of the end of the wharf would have been torn apart, and the damage would most likely have amounted to at least £200 to £250, and the ship may, as a result of the wharf giving way, have come off very lightly, especially if the propeller held.

In this particular case, however, the wharf held best; with the result that the tail-end shaft sheared off and the broken end with the propeller sank to the bottom of the dock.

Now compare the cost of repairing the damage to the wharf at £50, with the debit against the ship :—

	£
Cost of new tail-end shaft	350
Cost of new propeller, say £200, less allowance for the old one at £100 ...	100
Dry dock charges for removing old shaft and fixing new shaft and propeller, including docking dues, etc., say	350
Consequential delay to steamer—8 days at, say, £150 per day	1,200
Total ...	£2,000

Even neglecting for the moment the business cost due to delay, the cost of repairing the actual damage amounted to £800, as against £50 in the case of the wharf (*Verbum sat sapienti*).

DESIGN OF BEAMS, Etc.

This question of inertia brings me back to the original theme of the paper, *i.e.*, "Rational Design," and perhaps will serve to demonstrate where I sometimes differ with the specialists.

To revert to my previous statement regarding the wisdom of employing specialists, there is no doubt that under special conditions, where, for instance, amount of headroom or similar provision is of paramount importance, or some kindred important restrictions are in force, the

specialist, with his varied experience, is naturally best able to judge when to design and use double reinforced beams in preference to single reinforced ones, etc., but in the case of wharves of the kind we are at present discussing, where headroom, *i.e.*, depth of beams, is of no consequence, the same argument is applicable as I used in the case of the cylinders, *i.e.*, that weight is the great thing to provide, within reasonable limits, of course.

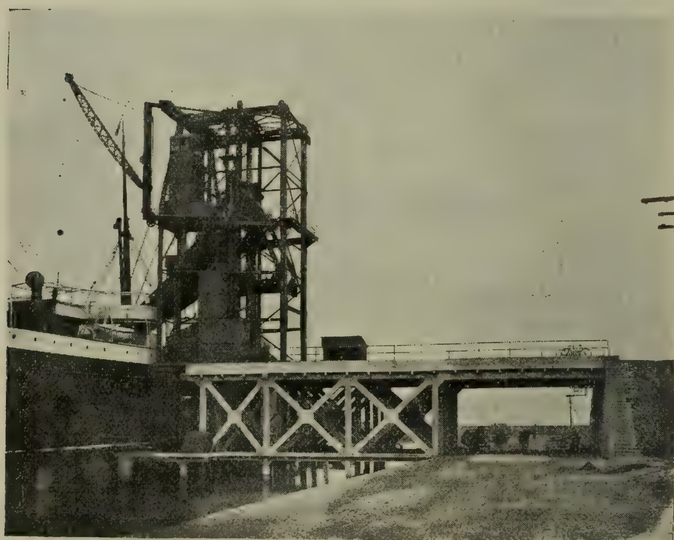

FIG. 12. REINFORCED CONCRETE WHARF AT PORT TALBOT.

I believe, therefore, in the wisdom of designing beams with single reinforcement in every possible instance as being by far the simplest to construct, cheapest as regards quantity of steel required (which is usually by far the most expensive item), and also as providing the greatest amount of concrete within economical ratios, and thereby providing as much dead weight as possible without unduly or extravagantly decreasing the ratio between dead and live loads, or spoiling the general æsthetic proportions of the structure.

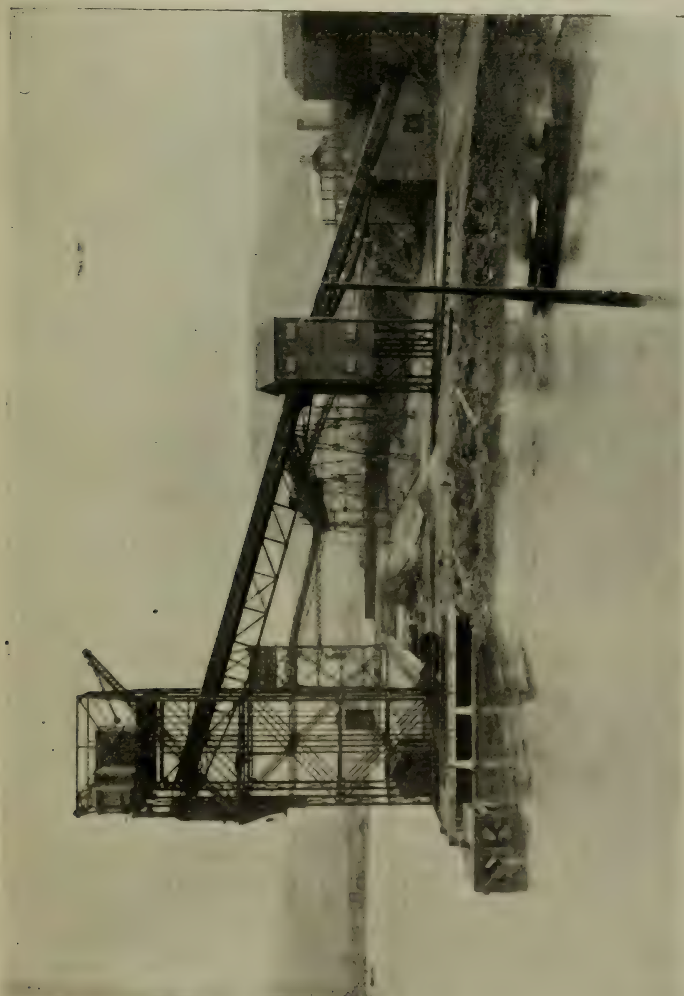

FIG. 13. REINFORCED CONCRETE WHARF AT PORT TALBOT.

SIZES OF STEEL.

Another point regarding calculations and their effect upon standardisation may be touched on. I believe in making preliminary calculations to as fine a point as possible, but where I disagree is in their application when getting out the working drawings. For bars of small section below $\frac{1}{2}$in. diameter, no doubt it is necessary to specify them in 1-16ths, but above $\frac{1}{2}$in. diameter I do not think it even economical in the long run, leaving out the question of convenience, to work to less divisions than $\frac{1}{8}$ths.

To design the entire reinforcement with all the rods varying by 1-16th makes it in the first place more difficult to procure them expeditiously from the mills, and afterwards adds very materially to the trouble of sorting, stacking, and subsequent selection for the work, coupled with the inevitable risk of placing the wrong size bars in various parts, owing to carelessness in taking measurements.

For instance, it is comparatively easy for a man who does not realise the responsibility to select 13-16in. bars in lieu of $\frac{7}{8}$in., or even 15-16in. bars for the same $\frac{7}{8}$in., with a resulting weakness in the one case and wastefulness in the other.

On the other hand, if the bars are graded in $\frac{1}{8}$ths, a man has to be grossly careless to make a mistake in measurement when selecting bars from a stock of that kind.

COVER OF CONCRETE.

In the past specialists have made a great mistake in designing beams with too little cover on the steel.

I am continually asked by those who are not in favour of reinforced concrete if I do not experience the trouble of the concrete peeling off the bars, due to the rust on same, causing internal expansion and bursting the concrete.

In reply, I am bound to admit that I have experienced this to some extent. In every instance, however, where I have found this it has been due either to defective workmanship or (as above stated) to what, from a commonsense practical standpoint, I consider too little cover provided in the original design, in proportion to the size of bar used.

If reasonable care is exercised to see that under no circumstances is there ever less cover of concrete at any place, top, bottom, or sides, than $1\frac{1}{2}$ times the diameter of the respective bars, usually adopted in beams (when above

a minimum size of, say, $\frac{3}{4}$in. diam.), I do not think the trouble of peeling or scaling of the concrete off the bars would ever be experienced, assuming, of course, that the concrete is of the proper quality.

Designers often serenely overlook the fact that the steel reinforcement when put together does not always lie in the strict plane they designed for it.

Under the best supervision and workmanship, the steel sags or buckles to some extent, and a fairly generous allowance in the concrete covering, factor of safety, and various other items, has to be made to compensate for these sundry unavoidable practical discrepancies.

In many respects I believe, therefore, in designing the work so that as many of the members as practicable can in the same way as the piles be manufactured individually and separately on the floor, then allowed to properly set, and afterwards built into the work as separate units, until subsequently the various junctions and intersections are completed, when the final work will be just as monolithic as if the job had entirely been concreted *in situ*, with the additional advantages enumerated below.

My reasons for constructing the members separately when possible are as follows :—The steel reinforcement can be laid out and built up, and the shuttering constructed, with far greater ease on the floor than in its final position, with consequently better facilities for inspection than when, for instance, the whole affair is suspended over the water.

Very often also the separate members, having set, are of very great assistance in supporting the remaining shuttering that has in any case to be carried out for various junctions, decking, etc.

There are, admittedly, occasions when the entire work can be much better done *in situ*, but what I wish to convey is, that in designing any job the question as to whether or not the various parts can be made separate should be borne in mind, or an attempt made to design the work with that end in view, as being the preferable method.

A very important point also in my opinion is to standardise the sizes of beams in each job as far as possible and to provide for the variation in stresses entirely by varying the sectional area of steel in each beam within practicable and reasonable limits, of course. A little careful thought in this respect on the part of the designer would often save the contractors and others a good deal of expense

in timber, etc., and possibly labour, too, in the making of the shuttering.

At the risk of being termed prejudiced, I believe in designing all work of the class under discussion with the proviso that mild steel bars only shall be used. Wharves and jetties are subject always to considerable shock, and high carbon steels are naturally much more brittle than mild steel, and subject to crystallisation through alternating stresses, much sooner; therefore I prefer to be on the safe side, even at a little extra cost, in the extra amount of steel required.

DETAILS OF FASTENINGS, Etc.

To revert to general design and to details of fastenings for bollards, etc., I believe in avoiding, wherever it is possible to do so, any Lewis bolts, or such fastenings which have to be permanently fixed into the work. All bolts, etc., should be so designed and fixed as to be quite getatable, if I may term it that way, and so as to be easily changed if broken or deteriorated to such an extent as to require renewal, and I therefore always arrange for pieces of wrought-iron tubing with internal dimensions to suit the respective diameters of the bolts to be provided, and these are always built permanently into the work as it proceeds.

This provides a clean and true hole in every instance, ready for the fixtures at any time, and without the bother and worry of having to bore or burn out wooden plugs, or anything of that kind, in accordance with the customary practice.

TRIAL PILES, Etc.

When designing wharves for sites where the constitution of the strata is unknown or uncertain, I would always recommend the driving of two or four trial piles, according to the size of the job, before any material for the piles is even ordered. This usually saves a great deal of trouble and anxiety in having to cut and lengthen the piles, etc., a rather unsatisfactory undertaking at the best of times, particularly if the piles have to be re-driven after lengthening, with the inevitable serious delay waiting for the lengthened portions to set. I always like to see the piles driven to within a few inches of their estimated level, and

the only cutting and stripping necessary being that required for the attachment of the beams, struts, etc. This avoids any joints being made near water-level, and this desirable end can be obtained only by the judicious driving of trial piles.

Whether the trial piles be of timber or concrete is to my mind not of material moment. Timber piles have the advantage, judged at any rate from my own experience, in the fact that they almost invariably drive a little further than concrete piles of the same section, so that any concrete piles made in accordance with the results obtained from a timber trial pile are always long enough, *i.e.*, on the safe side.

On the other hand, if concrete piles are available from some stock, they can, under certain restrictions, be driven, as trial piles, in position as permanent piles, and the cost of special piles thereby saved, but there is a risk in this from the contractor's standpoint by the fact that he must take the responsibility of having to remove the pile again if damaged, as the particular piles in question must of necessity be punished to a far greater extent as trial piles than would be the case with the remainder.

FIREPROOF QUALITIES.

I could no doubt go into many other questions *re* various aggregates for reinforced concrete, handling of cement, etc., but these do not strictly come under the head of design, and these points have no doubt in any case been well thrashed out in this Institute already. It may be well to mention one thing, however, which is to say that my primary reason for having taken up reinforced concrete in the first place was its undoubted fireproof quality. It may at first be wondered at why this should be considered so important in the case of wharves, etc., in docks with so much water available, and when the structure is absolutely built in the water. I would, however, point out that oil tank steamers often use these wharves, and are always liable to drain their tanks into the dock, however strict the regulations may be, leaving a dangerous film on the surface of the water surrounding the wharf. I have known such a film catch fire with almost disastrous results. Under such circumstances the fireproof quality of the reinforced concrete wharves is not only highly important as regards the actual

wharf itself as compared with, say, a timber one, but may also be the means of saving valuable material in adjacent warehouses, etc. In fact, one never knows the immense advantages that may accrue from the fact that a reinforced concrete wharf would, by not catching fire itself, prevent that fire spreading.

In order that the concrete wharves shall, in addition to not being inflammable, be also free from damage by fire, I insist on the rough aggregate being composed of granite, or blue pennant chippings, as being absolutely impervious to fire, and for this same reason will never permit limestone or similar chippings to be used.

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The Concrete Institute:

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, Etc.

Date of Reading: April 26, 1917.

A Method of Proportioning Materials for Concrete,

With the report of tests of the method compared with the usual style of giving proportions of materials.

By H. C. JOHNSON, M.C.I.

OUTLINE OF METHODS AND TESTS RESULTS.

A comparison is here made of the usual 1 : 2 : 4 method of proportioning concrete with one in which, no matter what the aggregates may be, the quantity of cement in the finished material remains the same for any given volume.

The percentage of dry cement, by volume, contained in the extremes of 1 : 2 : 4 concrete will vary all the way from 18% to as high as 24%.

The proposed method ensures a constant percentage of cement in the neighbourhood of 20-21%, and a specification based on the method would do at least two things:—

(*a*) Demand that exactly the same quantity of cement should enter a structure, whether it be built with gravel as the aggregate, or with broken stone as the aggregate. (Such would not be the case when it is specified that 1 : 2 : 4 must be the proportions.)

(*b*) Give a contractor or any other person the correct volume, and therefore the correct weight, of cement, that the volume of the concrete in the structure will demand.

Another part of the method is that some large aggregates are given less sand than others, therefore the ratio

of sand to large aggregate is not in the proportion of 2 to 4, but depends on the "workability" and weight per cubic foot of the concrete. It demands that actual tests to determine the proportions shall be made of those materials which it is intended shall go into the job, and therefore prevents an excess of fine stuff being used, as would otherwise be the case.

The tests show that the weight per cubic foot is increased from 2 to 3 lbs., while in one or two cases the addition in strength is as great as 500 lbs. per sq. inch at 28 days. In other cases there is an addition in strength even when less cement is used.

DETAILS OF METHOD AND TESTS.

It is well known that gravel and some other concretes, when mixed 1 : 2 : 4 are not as strong as hand broken stone concretes, also that the larger the aggregate, other things being equal, the stronger the concrete.

A. 1 : 2 : 4 gravel concretes are, on the average, about 12-15% weaker than stone concretes. They contain also, on the average, 10-15% less cement.

B. $\frac{3}{4}$ inch aggregate concrete is stronger than $\frac{1}{4}$ inch aggregate concrete, while $1\frac{1}{2}$ inch aggregate concrete is stronger than either. The $\frac{1}{4}$ inch aggregate has 500% greater surface for its volume than the $1\frac{1}{2}$ inch aggregate.

It was because of the above differences, A and B, that the writer was prompted to make the tests included in this paper, and the first outlet for results was found by writing an article, entitled, "What is a 1 : 2 : 4 Concrete?" which appeared in the February, 1915, issue of "Concrete and Constructional Engineering." In that article it was pointed out that by mixing concrete for two jobs of exactly the same size and volume, one, using gravel as aggregate, would call for 100 tons of cement, while the other, using broken stone as aggregate, would require 130 tons of cement. In spite of this there are a great many who believe that all 1 : 2 : 4 concretes contain the same amount of cement per unit volume, and therefore ought to be, in so far as cement content is concerned, of the same strength.

Since the article mentioned above was written to describe the results of tests made to determine the different

percentages of cements in 1 : 2 : 4 concretes in which different aggregates were used, and therefore thoroughly covers the ground leading up to the actual crushing tests mentioned later, the author proposes to use some parts of the article almost word for word.

It has been stated that there is a difference of 100 tons and 130 tons between the cement in gravel and stone concretes.

If that 24 per cent. can be saved by a contractor, it goes straight into his pocket, *plus the saving in not having to handle that quantity*; in other words, it is a *clear* profit, but a loss to the structure—a loss that seriously reduces its strength. This does not imply wickedness on the part of the contractor, but rather carelessness on the part of the architect or engineer.

The trouble is that the proportions 1 : 2 : 4, with too many people, stand for 2,000 lb. per sq. in. at 28 days. Some mixtures of 1 : 2 : 4 will allow that strength at that age—most will not come nearer than 1,500 lb. Such a statement does not condemn concrete—it condemns the makers.

For the present we will leave out the question of strength and consider only the amount, or, better still, the *volume percentage*, of cement in various 1 : 2 : 4 concretes. After all, the cement is the only thing that "cements," and the more of it, all other things being equal, the stronger the concrete.

Before going further I shall assume that you will admit the truth of the following facts:—

(*a*) Sand, stone and gravel are inert materials with absolutely no cementing power, and are only used to *dilute* neat cement, because neat cement would be too expensive to use alone. (This is quite apart from the question of using neat cement alone, since no one would use neat cement so.)

(*b*) Aggregates possess voids, or air spaces, ranging from 25 per cent. to 50 per cent. of the gross volume they occupy.

(*c*) It is incorrect to speak of a 1 : 2 : 4 mix as a 1 : 6 mix, since it may convey the impression that 6 parts of sandy gravel with 1 of cement is the same as 4 parts stone and 2 parts sand with 1 of cement.

(*d*) It is generally considered that a 1 : 2 : 4 mix gives a final volume of slightly over 4 parts, so that one-quarter of the mass is cement.

The following case illustrates the trouble caused by a loose specification. The architect called for "concrete to be composed of 6 parts of stone and sand, both measured by loose volume, to 1 part of Portland cement . . . by volume." The contractor was fortunate enough to find on the site a gravel of good quality, and asked the architect's permission to use it, which was readily granted. The contractor proceeded to mix 6 parts of the gravel with 1 of cement, when the architect stopped him, pointing out that the gravel should first be screened to remove the sand and then remixed in the proportions of 2 parts sand to 4 parts gravel. The contractor claimed he had a right to use 6 parts to 1 of cement. Neither side would give in, and the case went up for arbitration, when both admitted faults—the architect that his specification should have read 1 volume cement, 2 volumes sand, and 4 volumes stone; the contractor that he thought, when he was allowed to use the gravel, that 6 parts to 1 of cement was equal to 4 stone, 2 sand, and 1 cement.

It is very necessary to state that all measurements were made by *loose volume* (in the proportion 4 large aggregate, 2 small aggregate, to 1 cement) just as would be made on the job.

In order to obtain uniformity in proportions, the weights per cubic foot of the sands and the cement (the same brand being used for all mixes) were obtained at the start and used for mixing. All materials dry before mixing.

In summing up the points contained in the table no use will be made of Nos. 1, 8 and 9, because they are not 1 : 2 : 4 concretes in the strict sense, although many have been guilty of so considering them—guilty because even departures from rule-of-thumb methods, which are often satisfactory, must not be made to satisfy theory without first proving theory to be correct practically. The table illustrates the following facts:—

(*a*) 1 : 2 : 4 concrete No. 10 contains 29 per cent. more cement than 1 : 2 : 4 concrete No. 2.

(*b*) Stone concretes on the average contain 10 per cent. more cement than gravel concretes. It is therefore not

fair to expect gravel concretes to be as strong as stone concretes. ("American Civil Engineer's Pocket Book" notes that gravel concretes at one year are 9 per cent. weaker than stone concretes.)

(*c*) The percentage of voids in gravel being smaller than in stone, less sand should be used: 1½ parts sand (on the average) to 4 parts gravel will produce a concrete worked as easily as, if not more easily than, 2 sand to 4 stone.

(*d*) Gravels passed through and retained on screens as used for stone *always* have less voids; the popular idea is that they would be the same. It can be admitted that if stone could be "hand placed" into a receptacle, a very small percentage of voids would result. This indicates that stone concretes, unless they had considerable sand (2 to 4), would not be so reliable as gravel in the matter of density.

(*e*) Gravel concretes mixed 1 : 2 : 4 are 3 lb. per cubic foot lighter than stone concretes. Proper proportioning will easily make up this 3 lb., in spite of the fact that the specific gravity of gravel is only 2.51, as against 2.70 for stone.

I have used the terms *large aggregate* and *small aggregate* on purpose to draw attention to the fact that sand is only finer gravel or granite or quartz, is an inert material just the same as the larger materials, and has to be "cemented" by the cement just as the larger particles. It is no use talking of mortar filling the voids or anything of that sort. Mortar is used by bricklayers and masons to bed two plane surfaces together; concretors use inert aggregate and cement with which to bind it together; but the outstanding fact is that the *smaller the aggregate* (all other things being equal) the *weaker the concrete*. There is no need to quote authorities; they are many, and have established the fact. In view of this last, the less fine aggregate (sand), providing a thoroughly workable concrete is produced, the better.

Statements have been made, and tests devised to prove, that—

(*a*) Reducing the percentage of voids to the absolute minimum possible, and filling these with cement, produces the strongest concrete.

(*b*) Any cement beyond that necessary to fill these voids is waste and does not increase the strength.

(*c*) Just as a carpenter uses the minimum of glue in making a joint, so one should use a minimum of cement in making concrete.

Regarding (*a*), it is possible to reduce the percentage of voids to 10 by using a graded sand, but 10 per cent. of cement has to coat the surface of the remaining 90 per cent. Could this be the strongest possible concrete?

Regarding (*a*) and (*b*), a recent paper sought to prove that a mix of 1 : 2 : 4 was not stronger than one of 1 : 3 : 7. It *was not* at one month, but it was *over* 50 *per cent. stronger* at three months, and at a year would probably leave a greater gap. In the one-month test the 1 : 2 : 4 concrete contained more of, what was then, the weak material than did the 1 : 3 : 7, but at three months it began to assert its value, with the result stated.

Such statements would suggest that neat cement in compression would not be so strong as a mixture of very strong stone and cement. In two tests a first class neat cement cube gave 12,064 *lb*. per sq. in. at *three months*, another cube made of hard whinstone (probable crushing strength 20,000 lb. per sq. in.) $2\frac{1}{2}$ parts, and the same cement as above to fill the voids, one part, stood only 7,300 *lb*. per sq. in. at *eighteen months*.

It may be admitted, therefore, all other things being equal, the more cement the stronger the concrete. Gravel concrete has suffered for years, because for *apparently* equal quantities of cement it is not as strong as stone concrete.

The whole trouble lies in the fact that the amount of cement is not based on the finished volume of concrete.

GRAVEL AGGREGATES *versus* STONE AGGREGATES.

I speak of gravel concrete having suffered for years because when mixed 1 : 2 : 4 it is not usually so strong as broken stone concrete; this is because many engineers will not look at gravel as an aggregate if they can get broken stone. If I have succeeded in showing that it is not fairly treated in the matter of cement content, and if, later on, I also succeed in showing that it is unfairly treated in the

matter of sand in combination with it, I shall believe that it will be considered equally as good as, if not better material than, broken stone.

That it is as efficient as it is, is due to the fact that it adjusts itself and "flows" more easily than broken stone and is, therefore, the more reliable material where used by unskilled labour since it is less liable to form "pockets."

It would seem, from the above, that I am out in the interests of gravel aggregates, but if we were to confine ourselves to a comparison between hand or large broken stone and the usual form of machine crushed stone, I would feel called upon to defend the crushed stone in a somewhat similar manner, because it would have a smaller percentage of voids than the other and in a mixture of 1 : 2 : 4 would produce more concrete and therefore contain less cement. It is well known that the larger aggregates have higher strength—the greater cement content is one reason for this, while the fact that the surface area (to be cemented together) is less for an equal solid volume is the other reason.

LARGE AGGREGATES *versus* SMALLER AGGREGATES.

If we can, let us consider ourselves small enough to be able to wander about in the interstices of two different mixes of concrete—in one case a large aggregate concrete and the other a smaller aggregate concrete.

We would find in each case that small globules of air had attached themselves to the surfaces of all the materials making up the concrete, we would also find that owing partly to the "bridging" of all the particles and partly to insufficiency of very fine material there would be other passages and cul-de-sacs, then there would be some particles of cement engaged in sticking to the surface of the inert material, while still other particles would be sticking together in pairs and in threes and fours.

In the large aggregate concrete we would find that owing to the smaller surface area per unit volume of each piece there would be less cement adhering to the inert material and more cement particles in contact with one

another. There would also be a smaller total length of passages and cul-de-sacs.

Now I suggest we assume that each cement particle where it comes into contact with inert material has an *adhesive* value of one, but where it comes into contact with another cement particle it has a *cohesive* value of two or thereabouts. If then we have a smaller surface area per unit volume in one aggregate than another we must have a greater *cohesive* value and therefore greater strength.

The above would suggest the use of particles of aggregate as nearly spherical as possible, and while this, providing they were properly graded to fit into corresponding interstices, would be a reasonable thing to do, there still remains the fact that more or less slab-like material, if strong, has a reinforcing action and is, perhaps, of considerable value. On the other hand, this same slab-like material, if at all pyramid-like, may cause a wedging action under stress. It has been proved, however, in over 20,000 tests in America that there is no difference between "sharp" sand and sand of a spherical shape; of course the same may not apply in the case of the large aggregate. A test of concrete made of marbles, such as we used in our schoolboy days, with another using cube-like material, might throw some light in this direction and while unpractical would give valuable theoretical information.

Large aggregates should have large sand while smaller aggregates should have small sand. Even very fine sand, if of first quality, will give a concrete as strong as one with coarse sand if properly proportioned, *i.e.*, less of it used, which will be the case if the correct proportions are found by a trial mix in which one must aim at producing the *heaviest* concrete using those materials.

Having now decided two things—(*a*) that any concretes made up for comparative test values of different aggregates should all contain the same percentage of cement, and (*b*) that some large aggregates do not require so much sand as others, we (in the University Laboratory, Cork) proceeded to make up what we were pleased to call "Corrected Concretes" and also 1 : 2 : 4 concretes for comparisons of cement content and strength. We asked some of the stone and gravel merchants for materials and a cement company for cement, all of which were willingly supplied.

The following materials were used :—

Langstone Hb. ballast (Mr. Wentworth-Sheilds).
Co. Cork hand-broken limestone (purchased locally).
Co. Cork crushed whinstone (purchased locally).
Co. Kildare limestone gravel (Engineer, C.S. and W. Ry. of Ireland).
London ballast (Ham River Grit Co.).
Penmaenmawr granite (Penmaenmawr Granite Co.).
And a cement which shall be nameless.

We made up over 700 specimens and started to test at one month, when we discovered great variations, far greater than were expected. The cement had been tested by taking from one bag in four and it came out well, but upon testing the remainder of the bags individually we found no two bags alike. Certainly some were approximately the same in strength, but between the best and worst the following differences were shown :—

	7 days paste.	28 days paste.	7 days mortar.	28 days mortar.
Cement, Bag A	360	640	175	285
,, ,, B	470	630	240	380

Average of three in each case.

As a result of this we had to discard the whole of those 700 specimens. It was a serious lesson and I do not hide the fact, because it shows that although one may be assured by the cement seller that "it is all of the same consignment" (as was said in this case), it proves the necessity for continuous tests of cement supplies. They can all turn out good and also poor cement. Nevertheless, we are very grateful to those who so willingly supplied us with materials and at the same time very sorry that we have nothing reliable to place before them in return for their kind help.

In order to partly repair the damage we immediately started the making of over 400 more specimens, and the results of one and three months' tests are given in Tables 2 and 3.

TABLE I.

No. of Mix.	Large Aggregate and per cent. Voids.	Small Aggregate and per cent. Voids.	Loose Volumes.			Finished Volumes.	Weight per c. ft. Wet.	Cement in Finished Volume of Concrete		—
			Large Aggregate.	Small.	Cement.			By Ratio.	By per cent.	
	GRAVELS.						lb.			
1	Gravel as from river (33 per cent.)	None	6	—	1	6.0	118	1 to 6.0	16.70	
2	Gravel, $\frac{3}{4}''$-$\frac{1}{8}''$ (38 per cent.)	Sand from gravel, $\frac{1}{8}''$-$\frac{1}{76}''$ (40 per cent.)	4	2	1	5.6	140	1 ,, 5.6	17.90	
3	Gravel, $\frac{1}{2}''$-$\frac{3}{8}''$ ($42\frac{1}{2}$ per cent.)	Ditto	4	2	1	5.25	143	1 ,, 5.25	19.10	*Gravel Concretes* (excepting Nos. 1 and 8).—Average per cent. cement = 18.85; average per cent. voids = $38\frac{1}{2}$; average weight per cu. ft. = $143\frac{1}{2}$ lb.
7	Gravel, $\frac{3}{4}''$-$\frac{1}{2}''$ (43 per cent.)	Sand from gravel, $\frac{1}{8}''$-$\frac{1}{20}''$ (44 per cent.)	4	2	1	5.10	144	1 ,, 5.10	19.60	
8	Gravel, $\frac{3}{4}''$-$\frac{1}{8}''$, less $\frac{1}{2}''$-$\frac{3}{8}''$ material ($35\frac{1}{2}$ per cent.)	Sand from gravel, $\frac{1}{8}''$-$\frac{1}{76}''$ (40 per cent.)	4	2 parts 2-1 mortar		4.96	142	1 ,, 4.96	20.20	
11	Washed gravel, $\frac{3}{4}''$-$\frac{1}{8}''$ ($34\frac{3}{4}$ per cent.)	Sand from gravel carefully washed, $\frac{1}{8}''$-$\frac{1}{76}''$ (42 per cent.)	4	2	1	5.26	144	1 ,, 5.26	19.00	
12	Ditto	Special sand quite clean, $\frac{1}{8}''$-$\frac{1}{76}''$ (35 per cent.)	4	2	1	5.38	$146\frac{1}{2}$	1 ,, 5.38	18.60	

	STONES.									
4	Crushed limestone, size as No. 2 ($45\frac{1}{2}$ per cent.)	Sand as for No. 2 ...	4	2	1	4.92	145	1 ,, 4.92	20.40	*Stone Concretes* (excepting Nos. 9 and 10).—Average per cent. cement = 20.50; average per cent. voids = $46\frac{1}{2}$; average weight per cu. ft. = $146\frac{1}{2}$ lb.
5	Crushed limestone, size as No. 3 ($48\frac{3}{4}$ per cent.)	Ditto	4	2	1	4.90	144	1 ,, 4.90	20.50	
6	Crushed limestone, size as No. 7 (48 per cent.)	Sand as for No. 7 ...	4	2	1	4.80	149	1 ,, 4.80	20.85	
9	Crushed limestone, size as No. 8 (45 per cent.)	Sand as for No. 8 ...	4	2 parts 2-1 mortar		4.4	145	1 ,, 4.4	22.70	
13	Washed crushed limestone, size as No. 11 (45 per cent.)	Sand as for No. 11...	4	2	1	4.83	$148\frac{1}{2}$	1 ,, 4.83	20.70	
14	Ditto	Sand as for No. 12...	4	2	1	5.00	$149\frac{1}{2}$	1 ,, 5.0	20.00	
10	Crushed whinstone, $\frac{3}{4}''$-$\frac{1}{2}''$ (46 per cent.)	Fine sand, $\frac{1}{20}''$ - $\frac{1}{16}''$ (48 per cent.)	4	2	1	4.33	144	1 ,, 4.33	23.20	

TABLE II.

6″ Cubes.

Values given are average of three cubes in each case.

	Materials.	1:2:1 Concrete.				Corrected Concretes.				
		Per cent. Cement.	Wt. lbs. per cu. ft.	Strength lbs. sq. in.		Per cent. Cement.	Wt. lbs. per cu. ft.	Strength lbs. cu. in.		Proportion.
				1 mo.	3 mo.			1 mo.	3 mo.	
GRAVELS.	Sandstone gravel and fine sand	21.6	143½	2,400	3,360	20.8	146	2,575	3.575	1 : 1.43 : 4.5
	Ditto and coarse sand...	20.45	146	2,640	3,255	20.4	145½	2,292	2,970	1 : 1.43 : 4.3
	Limestone gravel and fine sand	21.85	142½	2,320	3,645	20.90	145	2,890	4,020	1 : 1.43 : 4.3
	Ditto and coarse sand...	21.35	144	2,675	3,165	21.0	146	3,060	3,970	1 : 1.6 : 4.15
STONES.	Crushed sandstone and fine sand	23.9	144	2,380	3,185	20.8	146½	2,750	3.355	1 : 1.4 : 4.6
	Ditto and coarse sand...	21.5	146½	2,920	3,718	21.75	146	2,730	3,650	1 : 1.8 : 4.4
	Hand broken limestone and fine sand	23.35	147½	2,450	3,110	21.0	148½	2,710	3,660	1 : 1.72 : 4.75
	Ditto and coarse sand...	22.25	149	2,535	3,103	21.0	148½	2,490	3,213	1 : 1.67 : 4.5

Cubes in moulds 48 hours, in water five days, in Laboratory air until broken.

Of fine sand two-thirds passed $\frac{1}{50}$″ sieve, all passed $\frac{1}{32}$″ sieve.

Coarse sand was Leighton Buzzard $\frac{1}{10}$ to $\frac{1}{40}$ about.

Water used averaged about 10% (see Table 6).

TABLE III.

3.16″ Cubes (10 sq. in. per face).

Values given are average of three cubes in each case.

Kind	Materials.	1:2:4 Concretes. Per cent. Cement.	Wt. lbs. per cu. ft.	Strength lbs. cu. in. 1 mo.	3 mo.	Corrected Concretes. Per cent Cement.	Wt. lbs. per cu. ft.	Strength lbs. cu. in. 1 mo.	3 mo.	Proportion.
KIND.	Sandstone gravel and fine sand	Same as for the 6″ cubes.	Same as for the 6″ cubes.	2,615	3,070	Same as for the 6″ cubes.	Same as for the 6″ cubes.	3,100	3,785	Same as for the 6″ cubes.
	Ditto and coarse sand...			2,880	3,645			2,675	3,680	
	Limestone gravel and fine sand			3,060	3,480			3,520	3,885	
	Ditto and coarse sand...			2,995	3,710			3,405	3,860	
	Crushed sandstone and fine sand			2,816	3,026			2,880	3,220	
	Ditto and coarse sand...			2,070	4,080			3,030	3,865	
	Hand broken limestone and fine sand			3,445	3,870			3,070	3,685	
	Ditto and coarse sand...			3,280	3,650			3,060	3,185	

(N.B.—These cubes are too small because there is a tendency to place more of the finer parts of the concrete in them in order to get it into the moulds easier. The moulds were not made for concrete but for paste and mortar tests of cement.)

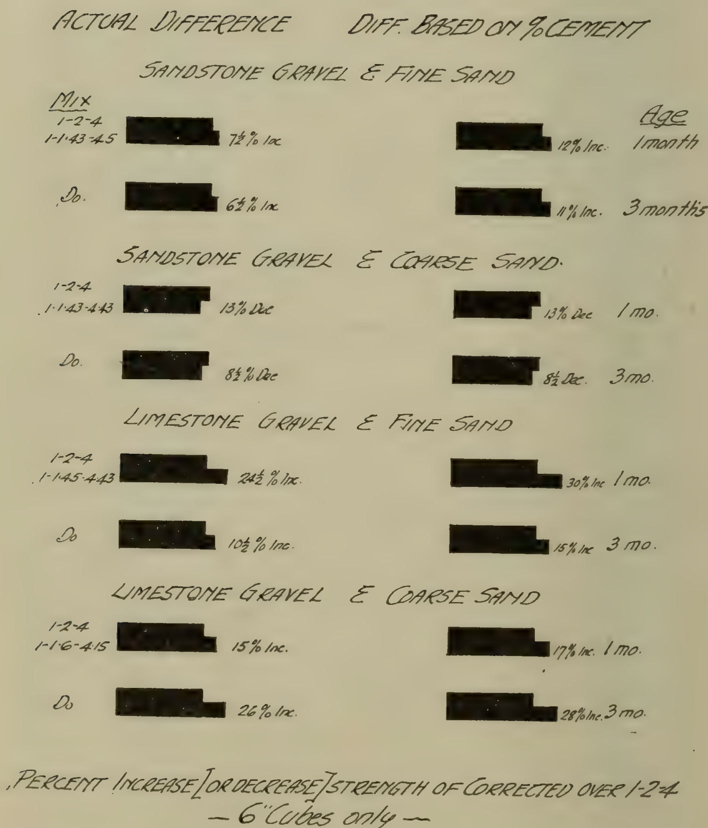

ACTUAL DIFFERENCE
DIFF. BASED ON % CEMENT
SANDSTONE GRAVEL & FINE SAND
Mix
Age
1-2-4
1-1·43-4·5
7½ % Inc
12% Inc.
1 month
Do.
6½ % Inc
11% Inc.
3 months
SANDSTONE GRAVEL & COARSE SAND.
1-2-4
1-1·43-4·43
13% Dec
13% Dec
1 mo.
Do.
8½ % Dec
8½ Dec.
3 mo.
LIMESTONE GRAVEL & FINE SAND
1-2-4
1-1·45-4·43
24½ % Inc.
30% Inc
1 mo.
Do
10½ % Inc.
15% Inc
3 mo.
LIMESTONE GRAVEL & COARSE SAND
1-2-4
1-1·6-4·15
15% Inc.
17% Inc.
1 mo.
Do
26% Inc.
28% Inc.
3 mo.
PERCENT INCREASE [OR DECREASE] STRENGTH OF CORRECTED OVER 1-2-4
— 6" Cubes only —

DIAGRAM 4.

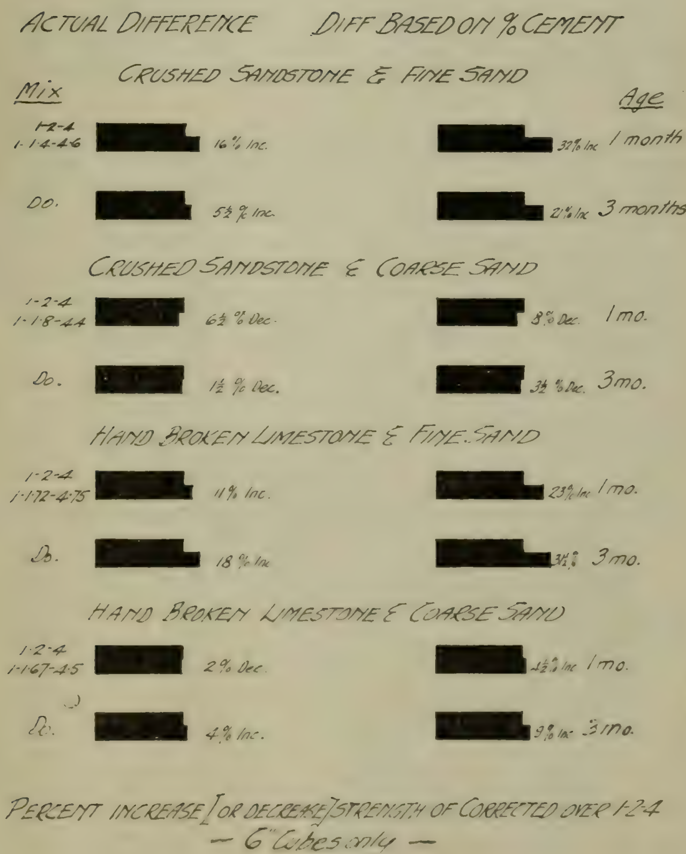

ACTUAL DIFFERENCE
DIFF BASED ON % CEMENT
CRUSHED SANDSTONE & FINE SAND
Mix
Age
1-2-4
1-1·4-4·6
16 % Inc.
32% Inc
1 month
Do.
5½ % Inc.
21% Inc
3 months
CRUSHED SANDSTONE & COARSE SAND
1-2-4
1-1·8-4·4
6½ % Dec.
8% Dec.
1 mo.
Do.
1½ % Dec.
3½ % Dec.
3 mo.
HAND BROKEN LIMESTONE & FINE SAND
1-2-4
1-1·72-4·75
11% Inc.
23% Inc
1 mo.
Do.
18 % Inc
3 mo.
HAND BROKEN LIMESTONE & COARSE SAND
1-2-4
1-1·67-4·5
2% Dec.
1½% Inc
1 mo.
Do.
4% Inc.
9% Inc
3 mo.
PERCENT INCREASE [OR DECREASE] STRENGTH OF CORRECTED OVER 1-2-4
— 6" Cubes only —

DIAGRAM 5.

MATERIALS USED IN THE TESTS.

For these specimens we mixed two brands of cement—

(*a*) South Wales P.C. Co.'s,
(*b*) Magheramorne P.C. (A.P.C.M.),

both of which were supplied free, the South Wales by the Company's Manager (Mr. Cooper), and the Irish cement by Messrs. Norman Macnaughton, of Cork, Dublin and Belfast.

The mixing was done by placing the whole of the cement into an Olsen "rattler" (used for testing road metal) and giving it 250 turns, the gaps in the "rattler" being temporarily stopped with pieces of wood.

The separate and combined tests of the cements were as follows:—

	Fineness.		Cement Paste.		Cement Mortar.		Per cent. Water.	
	76	180	7 days.	28 days.	7d.	28d.	Paste.	Mortar.
South Wales P.C....	.2	7.47	650	715	364	430	21.25	8.33
Magheramorne ...	.21	8.67	470	620	233	400	22.4	8.5
Combined	—	—	545	670	285	417	—	—

Average of three in each case.

Three of the aggregates mentioned above were used in the new tests in addition to a sandstone gravel obtained in Cork County.

We therefore had the following aggregates:—

Co. Cork sandstone gravel.
Co. Kildare limestone gravel.
Co. Cork crushed sandstone.
Co. Cork hand-broken limestone.
Co. Wexford fine quartz sand.
Leighton Buzzard sand, kindly supplied by Joseph Arnold, of Kentish Town.

All large aggregates were washed and passed over the following screens:—$\frac{3}{4}$, $\frac{5}{8}$, $\frac{1}{2}$, $\frac{3}{8}$, $\frac{1}{4}$, $\frac{1}{8}$, and only material passing the $\frac{3}{4}$in. and retained on the $\frac{1}{8}$in. screen was used.

The fine sand was so fine that two-thirds of it passed the $^1/_{50}$in. sieve. The Leighton Buzzard sand was the run

of the pit and varied between $^1/_{10}$in. and $^1/_{40}$in.; it is really a medium sand, but we called it our coarse sand since the other was so fine.

Each aggregate was made up with both fine and coarse sand, thus giving eight sets of specimens.

PERCENTAGE OF CEMENT USED IN CORRECTED CONCRETES.

Having made up "corrected" concretes in the 700 specimen series we were able to decide upon the percentage of cement to use in this, the second, series, and decided upon 21% where the cement was assumed to weigh 100 lbs. per cubic ft., which we found to be roughly the case, since a bag holds (when the cement has settled in it) about $2\frac{1}{4}$ cubic ft.

The average percentage of cement in 1 : 2 : 4 concretes is about 20 to 21.

ORDER OF MAKING SPECIMENS.

The 1 : 2 : 4 concrete of each material was made up first and immediately afterwards the "corrected" concrete was made, thus avoiding any possibility of a change in the cement's value, small though it might be after a few days.

METHOD OF PROPORTIONING.

The weights per cubic ft. of all materials were first found by getting the average of three tests, all loosely filled into the measure without any shaking or ramming, then in proportioning so many pounds of the required material was taken (corresponding to so many cubic ft. or parts of a cubic ft.), thus avoiding any errors of judgment in filling measures in using by volume.

The materials for the 1 : 2 : 4 concretes were, of course, easily weighed, but the "corrected" concrete materials had to be proportioned to get maximum weight per cubic ft. This work, however, is quite simple. We know that in order to make 1 cubic ft. of concrete we must have .21 cubic ft. of cement = 21 lbs. We also know that by trial and error we can arrive at the correct ratio of sand to large material, *but only by mixing the cement and water*

with them; in other words, *no proper concrete can be correctly made except by an actual test of the materials to be used.*

The following method and example will explain the making of a corrected concrete, using, in this case, 20% of cement. (The average of all 1 : 2 : 4 concretes.)

Take a ½ cubic ft. pail and measure .40 cubic ft. large aggregate and turn this on to mixing board, then measure .10 cubic ft. cement carefully. If large aggregate is gravel, measure out .125 cubic ft. sand (if stone, .15 cubic ft.) and turn these on to mixing board and mix with water—fill into other pail, noting volume and weight. This will not usually produce ½ cubic ft. or a pailful (which is necessary to obtain the correct percentage for the amount of cement used), so continue adding sand in small lots carefully measured until a *working consistency* is obtained, after which weigh and note volume; if pail is not full, add large aggregate, noting the volume, until pail is full. The last figures obtained give the proportions for the mix.

EXAMPLE.

	Volume.	Weight. lb.	Weight per cu. ft.
Gravel .4 cu. ft.+.1 cement+.125 sand ...	.435	62	144
,, .4 ,, +.1 ,, +.15 ,, ...	.460	67	145½
,, .4 ,, +.1 ,, +.175 ,, ...	.488	71	145½
(This produces a workable concrete, therefore add more stone.)			
,, .425 ,, +.1 ,, +.175 ,, ...	.502	73½	146½

N.B.—No account need be taken of the water used, but a trifle more than sufficient should be used in first batch and none added after.

The correct mix, then, is .425 large aggregate = 4¼, .175 small aggregate = 1¾, .100 cement = 1. Of course, these figures will only apply to the concrete using this sand. Another sand, with the same gravel, might produce the proportions 1 : 1.6 : 4, but there is still the same expenditure of cement for the same *finished* volume of concrete. Smaller proportions do not indicate smaller finished volumes.

Using this method, the two following concretes were produced, using the 14 per cent. wetted cement = 20 per cent. by loose volume :—

A.—Gravel 4¼ parts, fine sand 1½ parts, cement 1 part. Weight per cubic ft., 148 lb.

B.—Same gravel 4 parts, coarse sand 1.6 parts, cement 1 part. Weight per cubic ft., 146 lb.

Two others, using stone and the above sands, called for proportions so near 1 : 2 : 4 (one was No. 14 in table) that nothing would be gained by altering those proportions. This would not prove that stone concretes need not be proportioned, since only one size stone was used in these two mixes.

1 : 2 : 3	concretes	would	average	about 24%	cement.
1 : 1½ : 3	,,	,,	,,	27½%	,,

Correct specification for concrete equal to average 1 : 2 : 4 concrete—*i.e.*, containing 20 per cent by loose volume of dry cement equal to 14 per cent. wetted :—

The concrete shall be composed of perfectly clean crushed stone or gravel passing the -in. sieve and retained on the -in. sieve (⅛in. suggested); the particles of gravel shall have a least dimension of not less than one-half the greatest dimension, the stone a least dimension not less than one-third the greatest dimension, and no flakes that can be levered with a knife from the surface of any of the particles. Specific gravity not less than 2.5.

The sand shall be thoroughly washed, and when *dry* shall pass the ⅛in. sieve and be retained on the 1/76in. sieve, the particles shall be as nearly spherical as those of the gravel mentioned above. Specific gravity not less than 2.5.

The cement shall *easily* pass the minimum requirements of the British Standard Specifications of August, 1910, when not less than 22 per cent. of water is used for neat cement specimens and not less than 8½ per cent. for mortar specimens.

The proportions for mixing shall be determined on the principle that the minimum amount of sand consistent with easy working of the mass shall be introduced (not more than will give a ratio of 1 sand to 2 stone for stone concrete, or not more than will give a ratio of 1 sand to 2.4 for gravel concretes), and that the resultant thoroughly mixed concrete shall not contain less than 20 per cent. of cement by loose volume or 14 per cent. by wetted (22 per cent. water) volume. The weight per cubic ft. when wet shall not be less than 145 lb.

Samples of the materials proposed to be used shall be

deposited with the undersigned (architect or engineer) before any concrete materials arrive on the job, and the proportions to be used shall be proved to be as called for, by a demonstration in the presence of the undersigned, who will then give written authority to use such proportions if found correct.

Such a method of proportioning is easy for the contractor since he knows, as soon as he has his quantities out, exactly how much cement he requires.

For instance, if a job called for 100 cubic yards of concrete he would require 20 cubic yds. = 24 tons.

His quantities of large and small aggregate will be easily obtained as soon as a test has been made.

The architect or engineer will certainly know that his work contains a full and proper quantity of the only material which holds his building together.

METHOD OF MIXING, CURING AND STORING.

A Ransome mixer was used for mixing all specimens, and I may as well state that our average hand mixed concrete, using a good cement and similar aggregates, gave us only 1,400 to 1,700 lbs. per sq. in. at three months, which is very poor compared with the values of 3,000 to 4,000 lbs. in Tables 2 and 3.

The time of mixing in all cases was five minutes. This is longer than, probably upwards of three times as long as, on a job, but we wished to reduce to a reasonable minimum the unavoidable differences in time of putting the materials into the mixer, in fact all the human elements of error were reduced to the lowest possible.

The specimens were left in the moulds 48 hours, in water five days, and the remainder of the time in the Laboratory air. The temperature throughout varied between 54° and 62° F. They were stored so that only one face was unexposed at a time, that one upon which they rested, but owing to turning occasionally all faces received like treatment; a very necessary thing, since we found, in previous tests, that if piled one upon the other those in the centre were strongest, because of having dried out more slowly.

TABLE VI.

Percentages of water used in the concretes.

Materials.	1:2:1 Concretes.	"Corrected" Concretes.
	%	%
Sandstone gravel and fine sand ...	$9\frac{1}{2}$	$9\frac{1}{4}$
Limestone gravel and fine sand ...	$9\frac{2}{3}$	$9\frac{1}{3}$
Crushed sandstone and fine sand ...	$11\frac{1}{3}$	10
Broken limestone and fine sand ...	10	$9\frac{3}{4}$
Sandstone gravel and coarse sand ...	$9\frac{1}{4}$	$9\frac{2}{3}$
Limestone gravel and coarse sand ...	$9\frac{3}{4}$	$9\frac{2}{3}$
Crushed sandstone and coarse sand...	$11\frac{1}{4}$	11
Broken limestone and coarse sand ...	$9\frac{3}{4}$	$9\frac{3}{4}$

TABLE VII.

Test of 10-inch cubes of mortar, age 28 days, mixed 1 cement to 3 sand for sifted sand, others on % cement principal.

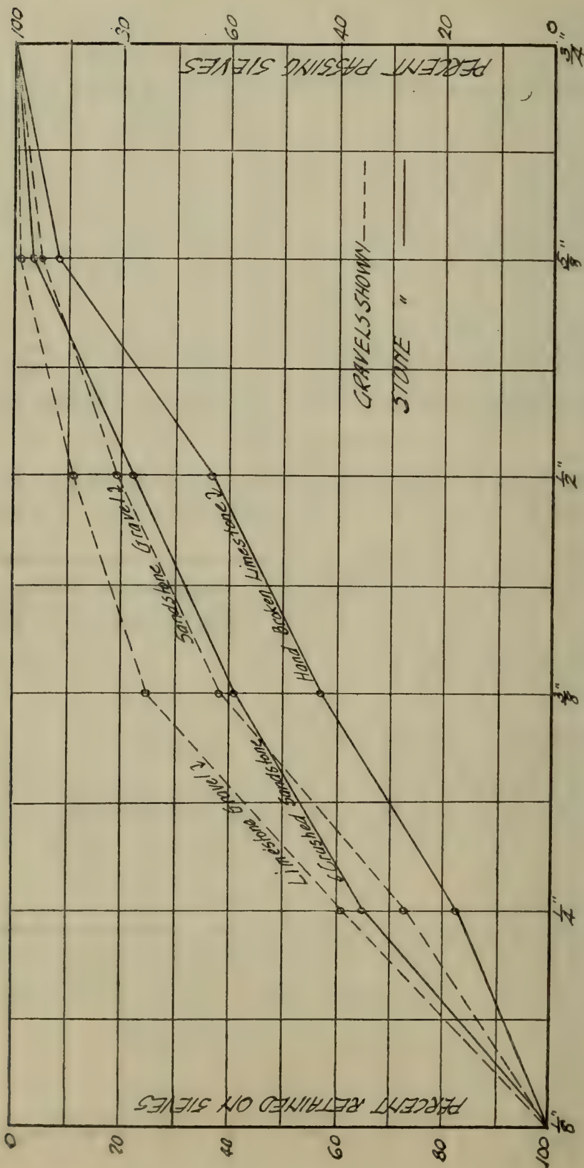
	Weight in oz.	Strength at 28 days comp. Individual in tons.	Average lbs. sq. in.
Fine sand mortar, Cube A	—	21.0	4,850
Cube B	40.5	22.3	
Sifted sand mortar, Cube C	42	26.0	5,620
Cube D	42	24.1	
Coarse sand mortar, Cube E	42.5	Held 30.4T. for $\frac{1}{2}$ min.	6,815
Cube F	42.5	Held 30.4T. for $\frac{1}{2}$ min. but unbroken, so removed and placed in larger machine giving 28.5T.	

All made on per cent. cement principal, *i.e.*, each cube had same quantity of cement in it.

Cement used was one of those used in tests, viz., South Wales.

METHOD OF TESTING.

Before breaking, all the cubes were weighed, measured, tested on four faces and ground on a dry plane surface if necessary (and it was generally necessary) to bring them to a plane face.



— GRANULOMETRIC ANALYSIS —

DIAGRAM 8.

They were then placed on a spherical seat in the Olsen 200,000 lb. testing machine, the upper platen brought down by the slowest drive and crushed without any lead, cardboard, or other packing.

The averages of three cubes in each case are given in Tables 2 and 3, while the percentage increase (in one or two cases a decrease) of corrected over 1 : 2 : 4 is given in Diagrams 4 and 5.

Diagram 8 gives the results of granulometric analysis of the large aggregates, while Table 6 gives the percentages of water used in terms of the total weight of all the dry materials.

Table 7 shows the effect of size of sand grains on the strength of mortar.

CONCLUSIONS.

1. The 1 : 2 : 4 method of proportioning should be considered obsolete, since no two 1 : 2 : 4 concretes contain the same percentage of cement, neither does it allow the majority of materials to produce their best values.

2. An *actual test* of the materials it is proposed to use should be made, introducing the percentage of cement required for the particular purpose the concrete is for and finding the ratio of small to large aggregate accurately by this means.

3. Other things equal, the percentage of cement closely governs the strength.

Other things equal, the larger the aggregate the stronger the concrete.

4. Previous tests proved that washing the average aggregate carefully will allow 30-40 per cent. higher strength in a hand mix, but only about 15-25 per cent. in a machine mix. This is always excepting really dirty material.

5. Using a mixer and giving 2-3 minutes for mixing will give a concrete, other things equal, about 50-75 per cent. stronger.

6. For equal working consistency and equal cement gravel concrete is as strong as stone concrete.

7. Gravel passing same screens as stone always has less voids than the stone.

8. Fine sand concrete has smaller weight per cubic ft. than coarse sand concrete.

9. Fine sand plus large aggregate (without cement) gives *smaller* volume than coarse. Fine sand plus large aggregate (with cement) gives *larger* volume than coarse.

10. Fine sand concrete is easier worked than coarse sand concrete for equal amounts of sand.

11. The finer the aggregate the more deleterious material and air it carries with it into the concrete.

12. The finer the sand the less should be used.

13. 30-40 per cent. higher strengths are obtained with 3.16in. cubes than with 6in. cubes.

14. Small cubes are more uncertain and inconsistent in the strength values than larger cubes.

15. In the future and in order that tests at various places shall be truly comparative, two things are required—

(*a*) The percentage of cement in the concrete.

(*b*) The strength of the cement in mortar tension or mortar compression.

I hope the discussion will now produce questions and opinions which will further add to any value this short paper, describing heavy work, may have, and it was heavy work, since we handled upwards of five tons of concrete, and now have as much respect for a cubic foot of finished concrete as a quantity surveyor has for a cubic yard.

Finally, I must place on record the great help I received from Messrs. W. Kingston, B.E., and D. M. Herphy, B.E., and also the encouragement and helpful suggestions we all received from Prof. C. W. L. Alexander, who, knowing the value of research, spared no pains to obtain for us the necessary apparatus with which to carry on the work.

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